CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

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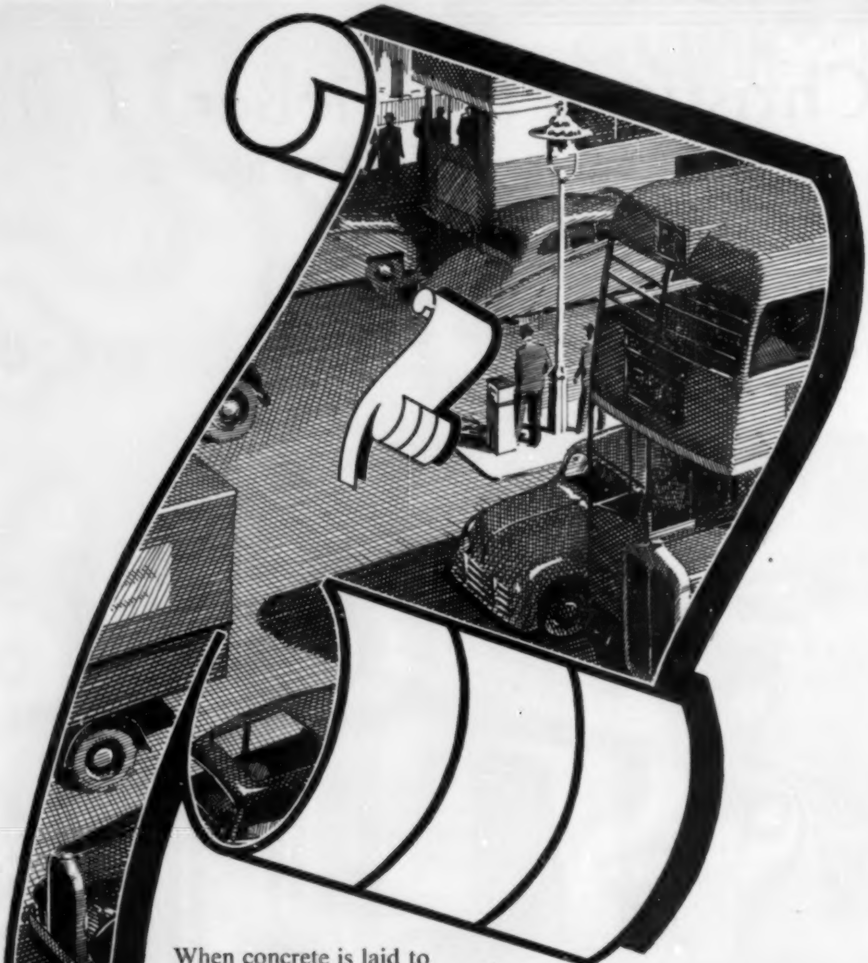

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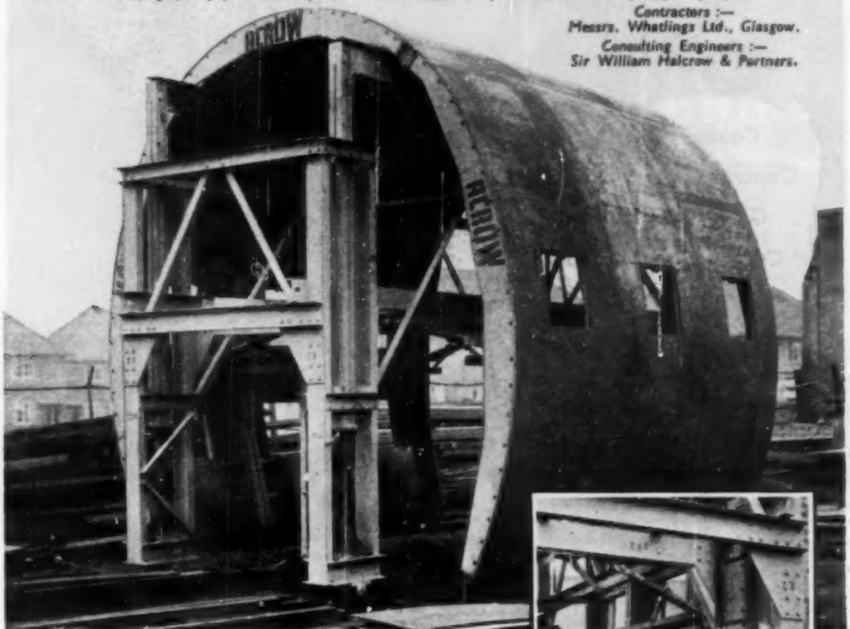

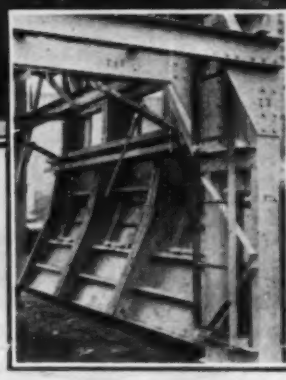

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CONCRETE
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Volume XLIX, No. 3. LONDON, MARCH, 1954

EDITORIAL NOTES

Road Works and Road Safety.

THE construction of new roads in the older civilisations has generally been the result of military necessity or a desire to provide employment. There have been exceptions, such as the roads built by Telford early last century for the post office in order to facilitate the mail, and the toll roads built and maintained by private enterprise in the eighteenth century and part of the nineteenth century. The Roman roads and the German autobahn are early and recent examples of highways built primarily for military purposes. In Great Britain the roads generally follow the ancient trackways, and were for long improved and kept in best repair when there were most unemployed men and women available to break stones. The many new roads built in the 1920's and 1930's had as an important objective the provision of work for the unemployed, who then numbered at times as many as three millions. Since 1945 there have been few unemployed and no strategic road requirements, so it is not surprising that little new road construction or major improvements have been done in the past few years in spite of the increasing volume of traffic.

The problem is beset with practical, financial, and political difficulties. At a time when there is little or no unemployment, more men can be put to work on roads only if they are taken from other employment ; the materials, machines, and tools required can be had only at the expense of other products of the engineering industry, and to-day the most urgent need for men and materials is in the manufacture of goods for export and for the equipment of our factories in order that we may be able to import the necessities of life. Finance can be provided only by the levying of more taxes or rates, and this at a time when the rate of taxation may discourage people from working longer or harder and also increases the cost of exports. The claim that the taxes on road vehicles and oil fuel should be spent on roads is quite unreal to-day. It is true that the original intention was that these taxes were to be used for the improvement of roads to meet the needs of the new traffic, but since the year 1936 the proceeds have been taken for the general purposes of government. The motor and oil taxes now produce nearly £300,000,000 a year, and the expenditure on roads is less than £100,000,000. If the whole of the revenue from these taxes were spent on the roads the difference of £200,000,000 would have to come from extra taxation, and however the money spent on roads be raised it would eventually be paid by the same people, at any rate so long as no political party when in power

seems able or willing to reduce expenditure in other directions. The political view of the question seems to be, as in the past, that road works are a useful standby in times of unemployment, and that it is not possible to provide men, materials, and machines for such work when there is none of them to spare.

The latest figures of the expenditure on roads are for the year 1951. In that year the expenditure on maintenance, repairs, improvements and new construction was £48,334,000, compared with £46,283,000 in the year 1939. Of these totals, £44,368,000 was spent in the year 1951 on repairs, maintenance and minor improvements compared with £29,684,000 in 1939. It is generally accepted that the cost of road works to-day, allowing for the greater use of machines, is about half as much again as in 1939, so it appears that the amount of actual work done on minor improvements and repairs in 1951 was about the same as in 1939. Between 1939 and 1952 the number of motor vehicles using the roads increased from about three millions to nearly five millions. However, whereas the number of people killed on the roads was 6648 in 1938, in 1953 this number had been reduced to 5070. For this there are many reasons, including better cars with more efficient brakes, more careful driving, more care on the part of pedestrians, improved pedestrian crossings, traffic lights, and so on.

It is probable, however, that one of the reasons for the fewer fatal accidents is the concentration on minor improvements and repairs and less on roads specially designed for high speed. Roads have been superelevated at dangerous bends, bottle-necks have been widened, "blind" corners have been opened out, and there are more warnings or roundabouts at dangerous crossings. Country roads which were used by pedestrians after dark only at great danger have been provided with footpaths. More reflectors, often showing the kerbs as well as the middle of the road, have been provided. Slippery surfaces have given way to surfaces which provide better adhesion for tyres. These improvements also lessen the time required for a journey, and it is suggested that in the interests of safety many more improvements to existing roads should be made before new roads are built if the money available is not sufficient for both. It is remarkable that with so many more vehicles on the roads and with about the same total expenditure there are fewer people killed. The fact that the density of traffic (that is motor vehicles per mile) on the roads in Great Britain is the highest in the world is an indication that more roads are needed, but the statistics available suggest that concentration on the improvement of existing roads has coincided with a welcome reduction in the number of deaths. The cost of traffic delays and the present rate of accidents must be reduced by all possible means—it is indeed lamentable that the number of people killed on the roads each month is greater than the total Servicemen from Great Britain reported killed during the whole period of the war in Korea. There seems, however, no doubt that the improvements carried out during the past few years, many of them of a minor and inexpensive nature, are helping to reduce the number of deaths on the road, and should be continued even when more men and materials are available for the construction of new roads, which will keep much of the traffic away from congested areas as well as saving time and money in road transport. The decision of the Government to spend more on new roads is very welcome, but it must not be allowed to detract attention from the need to spend still more money on minor improvements and in keeping existing roads in good repair.

Prestressed Multi-story Building in London.

THE office (*Fig.* 1) at Kilburn, London, for the Telephone Manager of the North-Western area of the London Telecommunications Region is a five-story framed structure 219 ft. long by 47 ft. wide, and provision has been made for an extension at the rear. The transverse structural frames are spaced at 12 ft. centres and consist of reinforced concrete columns supporting post-tensioned prestressed beams which span the width of the building. The prestressing cables were threaded through holes in the beams and columns and anchored on the outer face of the columns. Between these main beams, and spanning in the direction of the length of the building, are prestressed precast beams at 1 ft. 4 in. centres, between which are hollow clay blocks, the whole being covered with in-situ concrete to form a floor 6½ in. thick. This is the first multi-story continuous-frame structure embodying prestressed beams constructed in this country.

A cross section through the building is shown in *Fig.* 2. The main beams are rebated at the top, as shown in *Fig.* 3, to form a seating for the floor beams. Each main beam is prestressed by either one or two cables, the number of 0·276-in. high-tensile wires in each cable being shown in *Fig.* 2. The top story is 12 ft. less in width than the remainder of the structure, and the front columns are supported on the beams of the fourth floor. Rigid connections between these columns and beams would have necessitated the prestressing cables in the main beams having an impractical shape; therefore the bottoms of the columns of this story are hinged (see detail in *Fig.* 4). There is no mild steel reinforcement in the main beams other than that to resist the tensile forces near the soffit of the beams at the columns and some stirrups throughout their length to tie the floor slab to the beams. The longitudinal beams on the face of the

Fig. 1.—Front Elevation.

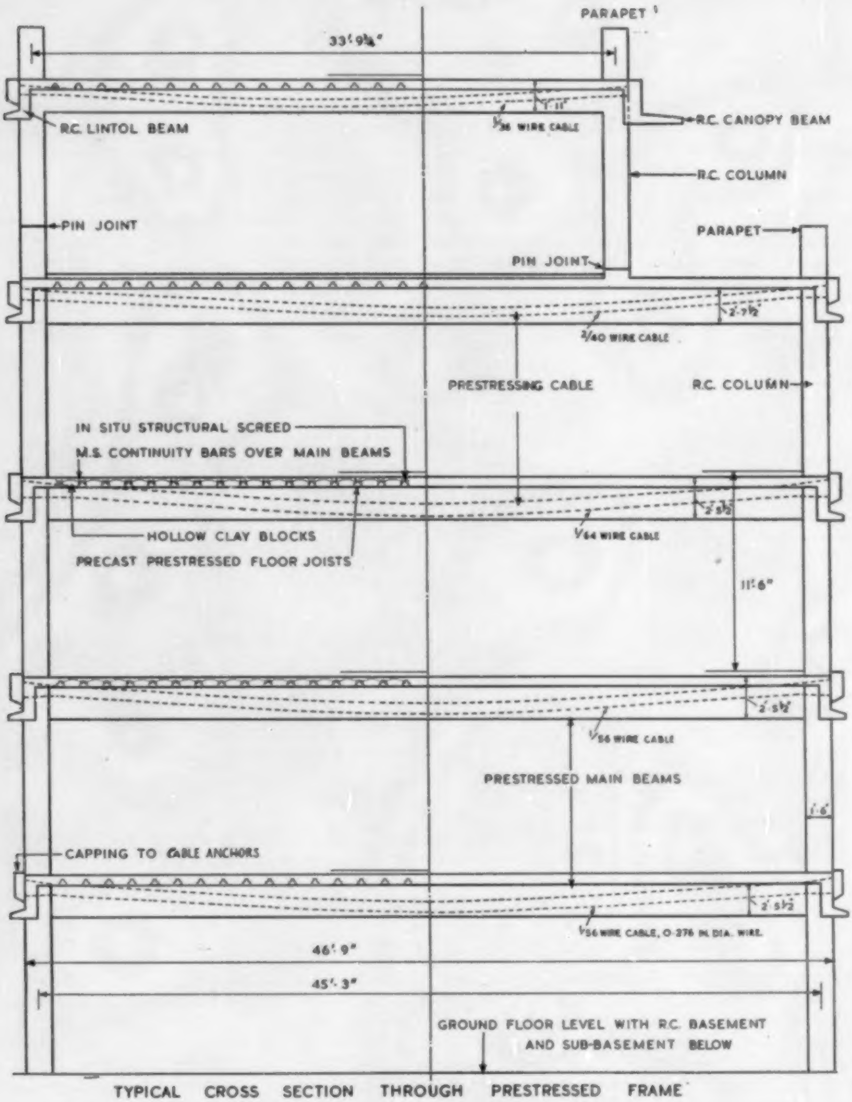


Fig. 2.—Cross Section.

building are of reinforced concrete ; these beams, the main beams, and the columns for each story were cast in one operation, a construction joint being formed in the columns 2 ft. above each floor level. The joint between the main beams and the columns is shown in *Fig.* 5.

The shutters to the sides of the main beams were removed twenty-four hours after the concrete was cast. The prestressed precast floor beams and hollow clay blocks were then laid and the floor concreted, except for a strip 1 ft. wide on each side of the main beams (*Fig.* 3), in order to have a maximum amount of dead load on the beams at the time of stressing. The main beams were prestressed at an age of fourteen days ;

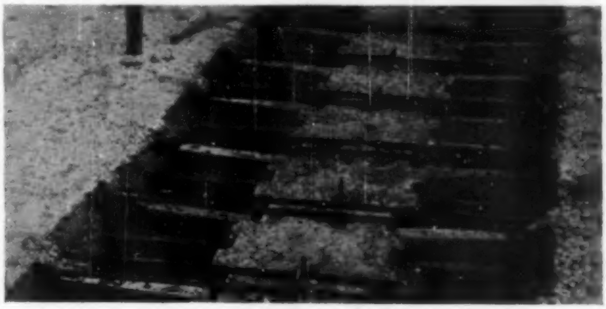


Fig. 3.—Main Beams for Floor Ready for Prestressing.

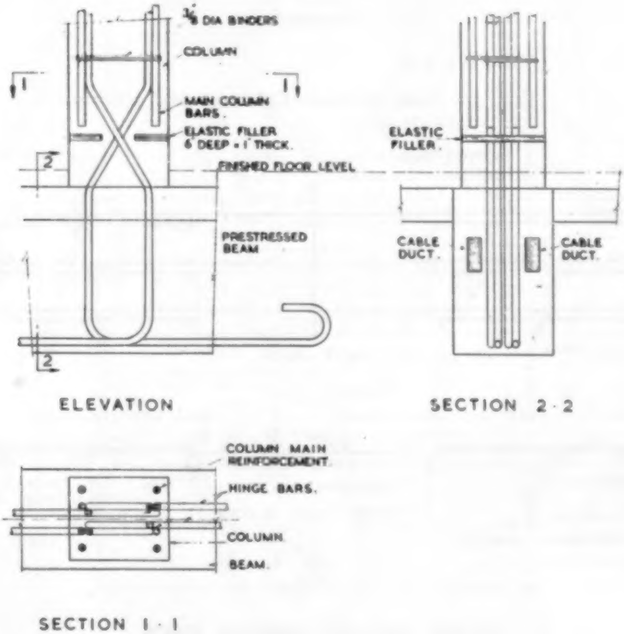


Fig. 4.—Hinges of Columns at Fourth-floor level.

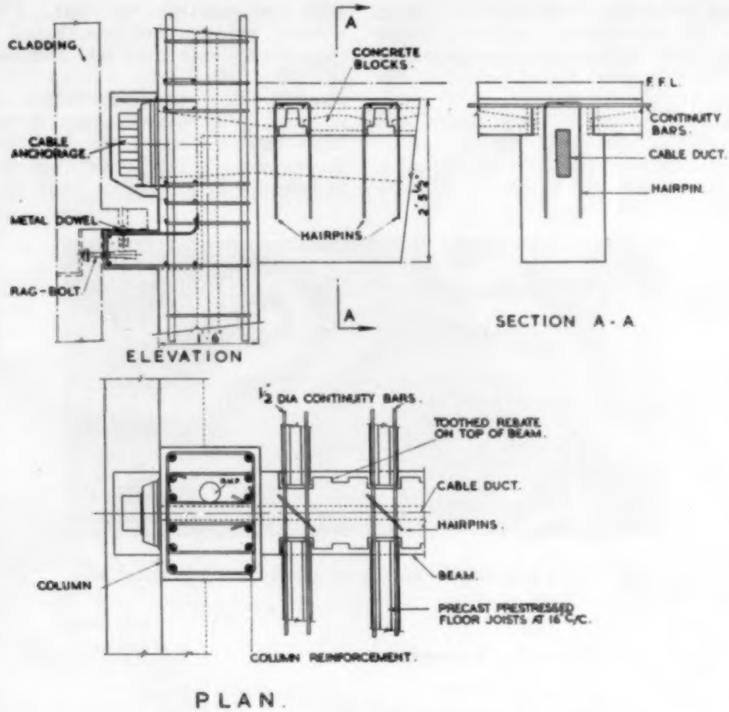


Fig. 5.—Joint between Columns and Beams.

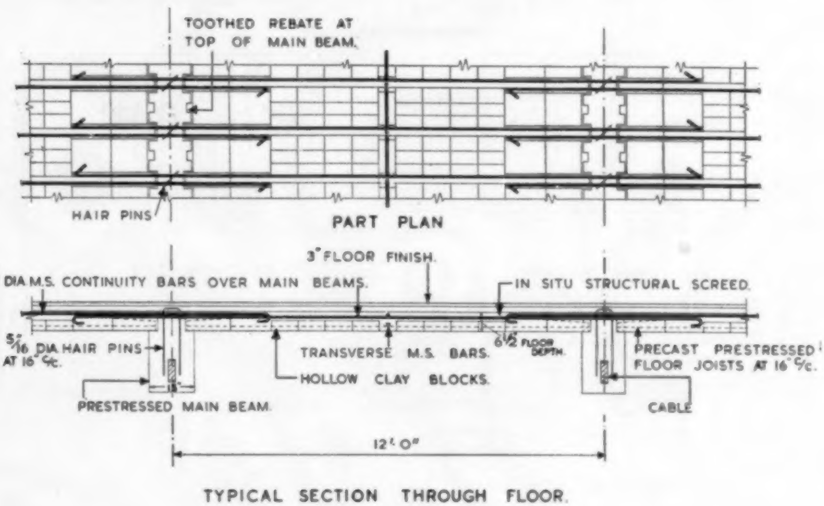


Fig. 6.—Details of Floor Slabs.

one-third of the wires in a cable were tensioned at a time to avoid producing high moments in the longitudinal beams along the sides of the building. The strips of in-situ floor adjacent to the main beams were then concreted, and formed flanges to the beams so that, when the live load is applied, the beams can act as tee-beams.

Mild steel bars are provided in the floors over the tops of the main beams to resist the reverse stresses due to continuity of the floor slab. Details of the floor slabs are shown in *Fig.* 6; they were designed for the following loads (in lb. per square foot): Imposed load, 50; finishes and ceiling, 35; partitions, 20. The total weight of steel in the floor slabs and main beams is 2 lb. per square foot of floor. The concrete was generally 1 : 1½ : 3, with a water-cement ratio of 0·4, but tests during construction showed that a 1 : 2 : 4 mixture was satisfactory. The crushing strengths of cubes at fourteen days were 6850 lb. and 5900 lb. per square inch respectively for the two mixtures. Wooden shutters were used with tubular scaffold struts.

The external walls are partly brick and partly precast concrete panels and mullions. The mullions, of story-height, are at about 4 ft. centres and are rag-bolted to the external longitudinal beams. The panels are about half the height of a story and are attached to the mullions by metal clips and bolts. Details of the

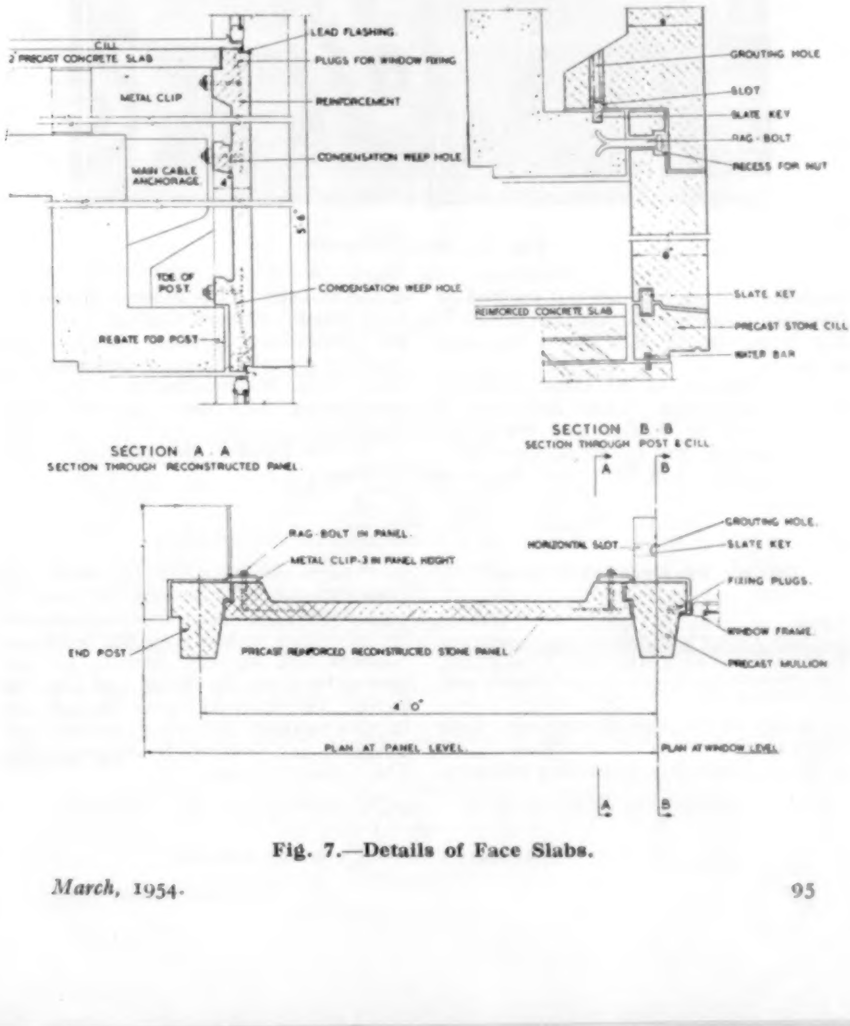


Fig. 7.—Details of Face Slabs.

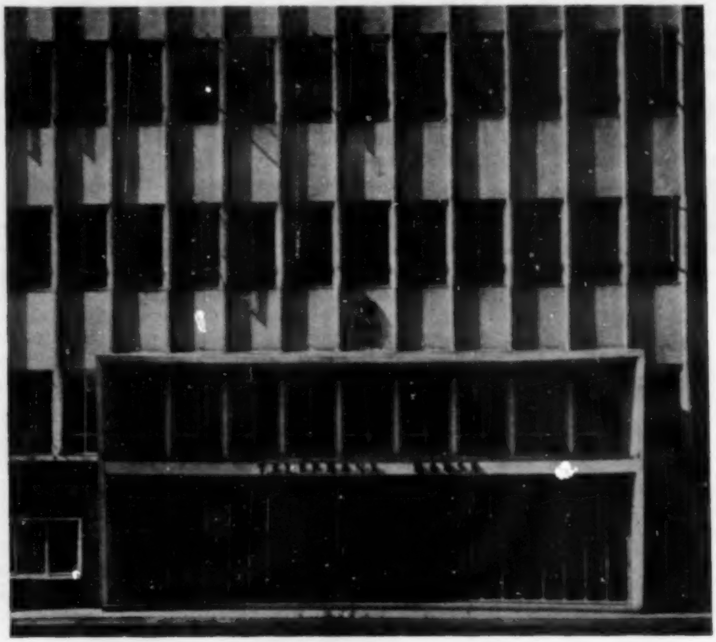


Fig. 8.—Main Entrance.

mullions and panels and the method of fastening them to the frame are shown in *Fig.* 7, and in *Fig.* 8 is shown the main entrance.

The architect is Mr. Eric Bedford, C.V.O., A.R.I.B.A., Chief Architect of the Ministry of Works, and the structural engineer Mr. G. C. A. Greetham, M.I.Struct.E., Chief Structural Engineer of the Ministry. The general contractors were Messrs. Richard Costain, Ltd., and the prestressed precast beams were supplied by the Costain Concrete Co., Ltd. The Magnel-Blaton system of prestressing was used for the main beams.

[A note on a test on this building is given on page 119.]

Patent Prestressed Beams.

FIG. I.

In beams or the like of composite form comprising a series of contiguous brick or like elements (1) constituting a bottom part on which concrete (4) containing prestressing wires (3) is cast, the elements (1) are provided with transverse members (2) to engage or key with the reinforced concrete and thereby increase the adhesion between the brick and concrete parts. The wires may pass through slots in the member (2).—No. 660,243. U. Bjuggren and Concrete Development Co., Ltd. June 7, 1950.

A Table for Calculating the Set of Piles.

By J. K. NESBIT, A.M.I.Struct.E.

A MODIFIED form [1] of the formula due to Mr. A. Hiley for calculating the safe load on driven piles is

$$F = \frac{wHen}{1 + cn} + w + p$$

where F is the load producing settlement (tons); w, weight of hammer (tons); H, effective fall of hammer (in.); e, efficiency of blow $\left(\text{depending on the ratio } \frac{p}{w}\right)$; n, number of blows per inch of final penetration; c, temporary elastic compression factor; p, weight of pile, helmet, and dolly (tons). The working load is then $\frac{F}{3}$ to $\frac{F}{1\cdot5}$, the lower denominator being for hard driving.

When the set is required to be known for a pile for which F is known, the equation is solved by assuming a value for n, substituting it in the formula, and repeating the procedure until the calculated and known values of F agree. If the piles are of different lengths, or F varies from pile to pile, or n has to be calculated for hammers of various weights, the solution can be tedious. However, if n is treated as the unknown quantity, and certain assumptions are made, the formula can be transposed and partially tabulated. Thus

$$\frac{1}{n} = \frac{wHe}{F - (w + p)} - c \qquad (1)$$

If d is the distance in inches for the last ten blows,

$$\frac{1}{n} = \frac{d}{10} \qquad (2)$$

If the hammer is winch-operated, and h is the actual drop in feet

$$H = 0\cdot8h \times 12 \qquad (3)$$

Substituting (2) and (3) in (1),

$$d = \frac{96whe}{F - (w + p)} - 10c \qquad (4)$$

The temporary elastic compression factor $c = 0\cdot5(c_1 + c_2 + c_3)$, where c_1 is the elastic compression of the pile, c_2 the elastic compression of the helmet and dolly, and c_3 the elastic compression of the ground. For a concrete pile driven into ballast with helmet and dolly the values of these factors given by Mr. Hiley are $c_1 = 0\cdot0134fL$ (in.), $c_2 = 0\cdot56f$ (in.) assuming the dolly and packing to be in good condition, $c_3 = 0\cdot22f$ (in.) where $f \left(= \frac{F}{A}\right)$ is the driving pressure in tons per sq. in., L the length of pile (in.), and A the cross-sectional area of the pile (sq. in.).

Therefore, $10c = 10 \times 0\cdot5(0\cdot0134fL + 0\cdot56f + 0\cdot22f) = \frac{F}{A}(0\cdot067L + 3\cdot9)$.

[1] "Reinforced Concrete Designer's Handbook." C. E. Reynolds.

TABLE I.

SETS REQUIRED FOR PILES DRIVEN INTO BALLAST OR CLAY.

BASED ON HILEY'S FORMULA (MODIFIED)

$$d = \frac{S}{F-(w+p)} - \frac{F}{T}$$

d = DISTANCE FOR LAST 10 BLOWS (IN.).
F = SETTLEMENT LOAD. (TONS).
w = WEIGHT OF HAMMER (TONS).
p = WT. OF PILE + HELMET + DOLLY (TONS).
S AND T TABULATED BELOW.

ASSUMPTIONS.
HAMMER IS WINCH-OPERATED.
PILE IS DRIVEN INTO BALLAST WITH HELMET AND PACKING IN GOOD CONDITION.
WT. OF HELMET ETC. EQUIVALENT TO 2'-0" LENGTH OF PILE.
WEIGHT OF PILE IS 155 LB. PER CU. FT.

VALUES OF S

PILE																										
12" × 12"		12'-6"	16'-0"	20'-0"	23'-6"	27'-0"	30'-6"	34'-0"	38'-0"	41'-6"	45'-0"	48'-6"	52'-0"	56'-0"	59'-6"	63'-0"										
14" × 14"			11'-0"	14'-0"	17'-0"	19'-6"	22'-0"	24'-6"	27'-6"	30'-0"	32'-6"	35'-6"	38'-0"	40'-6"	43'-6"	46'-0"	48'-6"	51'-0"	54'-0"	57'-0"	59'-6"	62'-0"				
16" × 16"					12'-0"	14'-0"	16'-6"	18'-6"	20'-6"	22'-6"	24'-6"	26'-6"	28'-6"	30'-6"	32'-6"	34'-6"	37'-0"	39'-0"	41'-0"	43'-0"	45'-0"	47'-0"	51'-0"	55'-0"	59'-0"	63'-0"
WT. OF HAMMER TONS.	DROP	p = 1.0	1.25	1.50	1.75	2.0	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00	4.25	4.50	4.75	5.00	5.25	5.50	5.75	6.00	6.50	7.00	7.50	8.00
1.5	2'-0"	177	164	153	144	135	127	120	113	107	103	99	96	92	89	86	83	80	77	75	73	72	68	63	60	58
	2'-6"	220	205	192	180	169	159	150	141	133	128	124	120	116	112	108	103.	100	97	94	92	90	85	79	75	72
	3'-0"	266	246	229	215	202	190	180	170	161	155	149	144	139	134	129	124	120	116	113	110	108	102	95	90	86
	3'-6"	310	288	268	250	235	222	210	199	188	181	174	167	161	155	150	145	140	136	132	128	125	118	110	106	101
	4'-0"	353	328	306	287	270	254	239	226	215	206	198	191	184	178	172	166	161	156	151	146	142	136	126	120	116
	4'-6"	396	360	345	323	303	285	269	258	241	233	223	214	206	199	193	187	181	176	170	165	161	152	142	136	130
2.0	2'-0"	265	245	228	214	204	194	186	177	169	161	154	147	142	138	134	130	126	125	120	117	114	108	104	100	96
	2'-6"	330	306	285	269	256	244	233	222	211	201	192	184	177	172	167	163	159	155	151	147	144	136	129	124	119
	3'-0"	398	366	342	322	305	291	278	266	253	241	230	221	213	206	200	195	190	185	180	176	172	162	155	150	143
	3'-6"	464	430	400	375	356	340	324	309	296	282	269	258	249	241	234	228	222	216	210	205	200	190	181	174	167
	4'-0"	530	490	456	429	408	388	370	353	337	322	307	294	284	275	268	261	254	247	240	234	228	217	207	199	190
	4'-6"	597	550	512	483	458	436	416	397	379	362	346	331	320	310	302	294	285	278	271	266	257	244	232	224	215
2.5	2'-0"		332	309	293	278	266	255	246	235	226	218	210	202	195	189	183	178	173	169	166	162	155	150	144	137
	2'-6"		416	386	365	348	333	319	306	294	283	272	262	253	244	236	229	222	217	212	207	202	194	187	180	172
	3'-0"		498	466	440	417	398	382	367	353	340	327	315	304	296	286	275	267	260	254	248	243	233	225	216	206
	3'-6"		580	540	512	486	464	444	428	412	396	381	368	355	342	330	320	310	304	296	290	283	272	262	251	240
	4'-0"		662	618	585	556	531	510	490	471	453	436	420	405	391	378	366	355	347	339	331	324	310	300	288	275
	4'-6"		747	698	660	625	596	572	550	529	509	490	473	456	439	426	412	400	390	381	372	364	349	336	324	309
3.0	2'-0"			397	376	357	342	328	317	306	296	286	277	269	261	253	245	237	230	224	218	213	205	197	190	184
	2'-6"			497	470	446	427	410	394	381	369	357	346	336	326	316	306	297	288	280	273	267	256	247	238	230
	3'-0"			597	564	535	512	492	474	458	443	428	415	402	390	378	367	356	346	336	327	320	308	296	285	276
	3'-6"			696	657	624	598	574	552	534	516	500	485	470	456	442	429	416	403	392	381	373	359	346	333	322
	4'-0"			795	760	714	684	656	632	610	590	571	664	538	522	506	490	475	460	448	436	426	410	395	380	369
	4'-6"			894	843	803.	769	738	710	686	664	643	623	603	585	575	551	534	518	504	493	480	462	445	428	415.

VALUES OF T FOR BALLAST

LENGTH OF PILE.	12'	14'	16'	18'	20'	22'	24'	26'	28'	30'	32'	34'	36'	38'	40'	42'	44'	46'	48'	50'	52'	54'	56'	58'	60'
12" × 12"	31.0	30.1	29.2	28.4	27.7	27.0	26.3	25.7	25.0	24.4	23.8	23.2	22.7	22.2	21.8	21.4	21.0	20.6	20.2	19.8	19.4	19.1	18.8	18.5	18.2
14" × 14"	41.7	40.4	39.3	38.3	37.3	36.4	35.5	34.7	33.9	33.2	32.5	31.8	31.1	30.4	29.7	29.1	28.5	27.9	27.4	26.9	26.4	26.0	25.6	25.2	24.8
16" × 16"	54.2	52.8	51.4	50.1	48.8	47.6	46.5	45.4	44.4	43.4	42.4	41.4	40.6	39.7	38.9	38.1	37.4	36.7	36.0	35.3	34.7	34.1	33.5	32.9	32.3

VALUES OF T FOR CLAY.

LENGTH OF PILE.	12'	14'	16'	18'	20'	22'	24'	26'	28'	30'	32'	34'	36'	38'	40'	42'	44'	46'	48'	50'	52'	54'	56'	58'	60'
12" × 12"	24.8	24.3	23.7	23.2	22.8	22.3	21.8	21.4	20.9	20.6	20.1	19.8	19.5	19.1	18.8	18.4	18.1	17.8	17.5	17.2	17.0	16.7	16.5	16.2	16.0
14" × 14"	33.8	33.0	32.2	31.6	30.9	30.3	29.7	29.1	28.5	28.0	27.4	26.9	26.4	26.0	25.5	25.1	24.6	24.2	23.8	23.5	23.1	22.8	22.4	22.1	21.8
16" × 16"	44.2	43.2	42.2	41.3	40.4	39.6	38.8	38.0	37.2	36.5	35.8	35.2	34.6	33.9	33.4	32.8	32.2	31.7	31.2	30.7	30.2	29.7	29.3	28.8	28.4

Substituting in (4), $d = \dfrac{S}{F - (w + p)} - \dfrac{F}{T}$, where $S = 96whe$ and $T = \dfrac{A}{0{\cdot}067L + 3{\cdot}9}$. If the pile is to be driven into clay $c = 0{\cdot}44f$ and $T = \dfrac{A}{0{\cdot}067L + 5}$.

For a given size and length of pile, T can be obtained. For a given fall and weight of hammer w and h are known, and e may be determined from the ratio $\dfrac{p}{w}$.[1] Thus the values of S may be tabulated as in *Table* 1, in which the values are given of p, S, and T for piles 12 in., 14 in., and 16 in. square and up to 60 ft. long, and for hammers weighing 1½, 2, 2½, and 3 tons with falls from 2 ft. to 4 ft. 6 in.

Examples.

1. The set is required for a pile 14 in. square by 38 ft. long to be driven into ballast by a hammer weighing 2½ tons and falling 3 ft.; F is 90 tons.

With $F = 90$ and $w = 2{\cdot}5$, from the table $p = 3{\cdot}75$, $S = 315$, and $T = 30{\cdot}4$.

Then $d = \dfrac{315}{90 - 6{\cdot}25} - \dfrac{90}{30{\cdot}4} = 3{\cdot}76 - 2{\cdot}96 = 0{\cdot}80$ in. for the last ten blows.

2. Calculate alternative falls with different weights of hammers for the pile in the preceding example. The required value of S is 315 for $p = 3{\cdot}75$. For a 2-tons hammer falling 4 ft. 6 in., $S = 331$, and for a 3-tons hammer falling 2 ft. 6 in., $S = 346$, either of which would be satisfactory with a set of 0·8 in.

3. Find the set required if an extension of 10 ft. 6 in. is cast on to the pile in Example 1. $F = 90$ and $w = 2{\cdot}5$ as before, and, from the table, $p = 4{\cdot}75$ for a pile 14 in. square by 48 ft. 6 in. long, $S = 275$ for a 2½-tons hammer falling 3 ft., and $T = 27{\cdot}3$.

Then $d = \dfrac{275}{90 - 7{\cdot}25} - \dfrac{90}{27{\cdot}3} = 3{\cdot}31 - 3{\cdot}29 = 0{\cdot}02$ in.

In this case either a greater fall or a heavier hammer is required. Thus for a 2½-tons hammer falling 4 ft., $S = 366$ and $d = \dfrac{366}{90 - 7{\cdot}25} - \dfrac{90}{27{\cdot}3} = 4{\cdot}42 - 3{\cdot}29 = 1{\cdot}13$ in.

(1) "Reinforced Concrete Piling." F. E. Wentworth Sheilds & W. S. Gray.

Lectures on Concrete.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

Soil Mechanics in the Building Industry, by I. K. Nixon. King's College, Newcastle-upon-Tyne. March 16. 7 p.m.

Modern Developments in Concrete, by R. C. Blyth. Technical College, Kingsway, Scunthorpe. March 17. 7.15 p.m.

Soil Mechanics Related to Foundation Construction, by M. W. Leonard. 1 Westmorland Road, Bromley, Kent. March 22. 7.15 p.m.

Problems of Plastering and Rendering, by E. L. Westbrook. Lecture Room, Central Library, Swansea. March 23. 7 p.m. And at the Central Café, Stepney Street, Llanelly. March 24. 7 p.m.

Tests on Prestressed Slabs.

MONSIEUR Y. GUYON, of Paris, writes as follows.

Mr. Andrew Gallia has asked me to comment on his article on "Design of Cantilever Slab Bridges with Stiffened Kerbs" which appeared in your October, 1953, number, but I do not think that I can add anything of interest from the point of view of elastic theory as the article is complete in itself. This article, and also the article on rigid frames by the same author in your December, 1952, number, are an extremely useful completion of the notable work of the University of Illinois on "Moments in Simple-span Bridge Slabs with Stiffened Edges" (Bulletin No. 97, April, 1939).

I have, however, recently carried out some tests on continuous prestressed slabs, an account of which will be published shortly. These tests indicate a very satisfactory agreement with the common theories at the elastic stage, and in particular with the work of the University of Illinois which has served as a check on my results. But it seems to me that the interest of these tests lies chiefly in the phenomena observed when the elastic stage has been passed. The principal facts noted were as follows.

(1) The tensile strength of the concrete was fully employed. While in the case of a beam it is necessary to take account of the minimum tensile strength because of the possibility of a local weakness, it seems from my tests that in the case of a slab the average tensile strength can be entirely taken into account. The explanation seems to be that a transverse crack in a beam extends to the sides of the beam, whereas a crack in a slab stops where it meets stronger concrete.

(2) Tensile plastic strains in the loaded zone give rise to normal forces. Lines of compressive forces are set up starting from the load and extending towards the supports; these stresses relieve the initial system, and a greater load can be imposed before cracks occur.

The tests were carried out on six continuous slabs measuring 3 m. by 1·25 m. by 8 cm. prestressed to 15 kg. per square centimetre (210 lb. per square inch) in both directions, and subjected to a single concentrated load (applied on an area 15 cm. square) on one of the slabs. According to the elastic theory cracking should have appeared at 800 kg. in the case where the tensile strength was zero, and at 2700 kg. in the case where the tensile strength was 38 kg. per square centimetre (a value obtained from tests on prisms). In fact a crack occurred with loads varying between 4100 and 4800 kg.; this crack was about one-hundredth of a millimetre (one two-thousandths of an inch) wide and was invisible to the naked eye.

The load was increased to about 8000 kg. without the crack opening wider. These cracks were along a line perpendicular to the short span over a width of about 1 m. Since these cracks are probably not deep, and will close on removal of the load, it is reasonable to estimate that a load of 8000 kg. is an acceptable working limit.

At loads from 8000 kg. to 16,000 kg. the width of the cracks increased and two additional cracks appeared along the diagonals of the slabs. Failure of two slabs occurred with loads of 22,000 kg. and 27,000 kg. It must be remembered that these were prestressed slabs, and that it is necessary for further tests to be made with different loads.

It would be rash to relate these results to reinforced concrete slabs. It is likely that a continuous reinforced concrete slab is much less sensitive than might be thought to local forces, and that redistribution of moments as a result of plastic phenomena causes the reinforcement to be effective over a much greater width than that considered in elastic theories. Also in reinforced concrete smaller loads will cause cracks than in the case of prestressed concrete. It is, however, possible that economies can be made in reinforced concrete slabs, and it is desirable that tests should be carried out.

Precast Construction in Denmark.—III.*

COMPOSITE PRECAST AND IN-SITU CONSTRUCTION.

By Søren Rasmussen.

Figs. 20 and 21 show the Kallton system in which precast members, in some cases with in-situ cores filled after erection, are connected by in-situ concrete joints.

The columns (*Fig.* 22) have projecting loops of steel reinforcement. The members are in pairs with staggered horizontal joints on opposite sides, and the core is filled with in-situ concrete which embeds the loops. Tests to failure show that the strength of such a column closely corresponds to that of an ordinary reinforced concrete column. The connections between beams and columns are formed in the joints between the precast members of a column, and the reinforcement at these positions is as is commonly used in reinforced concrete. The greatest story-height for which these columns have so far been used is 14 ft. The units are made in widths of 10 in., 11½ in. and 13½ in. This width determines one dimension of the column; the other cross-sectional dimension may be increased beyond 13½ in. by increments of

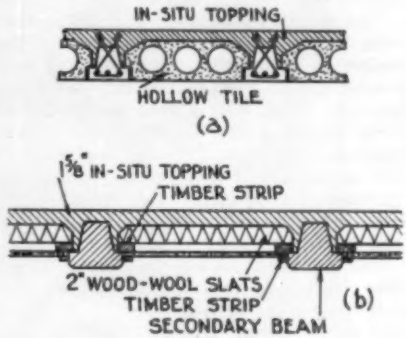


Fig. 21.

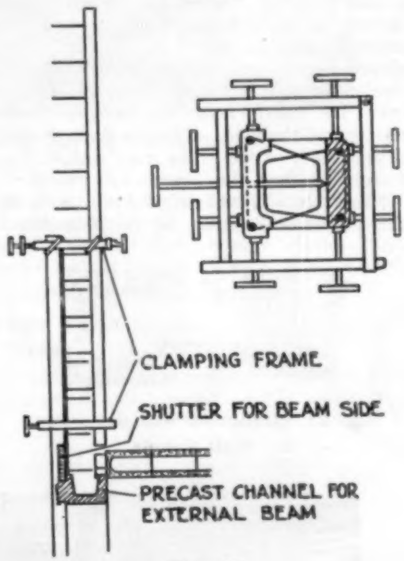


Fig. 22.

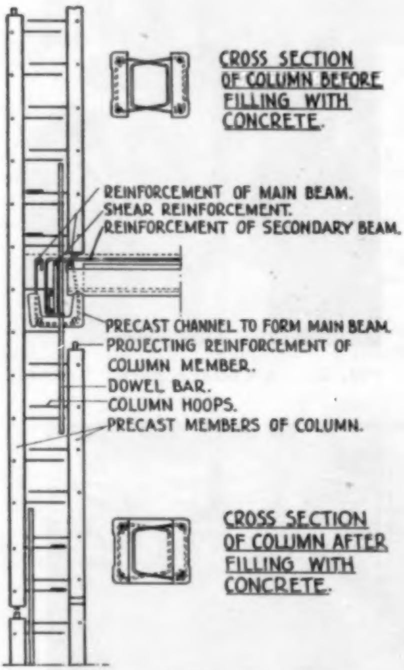


Fig. 20.—Precast Columns.

* *Concluded from February number.*

2 in., the maximum width used so far being 2 ft. 7 in. Columns may be built with different units, as shown in *Fig.* 23, so that recesses of 1 in. are formed to receive wall panels or the like.

The main beams are of channel section with hooked stirrups. The ends of the beams are connected to the columns with in-situ concrete when the trough of the beam is filled. As a rule the depth of these beams is the same as that of the channel section (6 in.) plus the floor or roof slab. If required the depth may be increased by lowering the level of the beam in relation to the top of the slab. The maximum span is about 21 ft.

On the main beams are placed secondary beams of inverted tee-shape with stirrups projecting from the top (*Figs.* 21 and 24). Precast hollow blocks rest on the flanges of these beams and a topping is cast in situ; the beams are supported during construction. The beams are made in lengths up to 26 ft. and weigh about 25 lb. per linear foot. Alternatively the spaces between the tee-beams may be filled with lightweight slabs rebated at the sides so that a flush ceiling is provided, or the slabs may rest on the flanges so that the beams are visible.

ERECTION.—Erection is done, without the use of scaffolding, by hand-operated

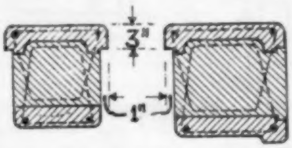


Fig. 23.—Columns with Recesses for Wall Panels.

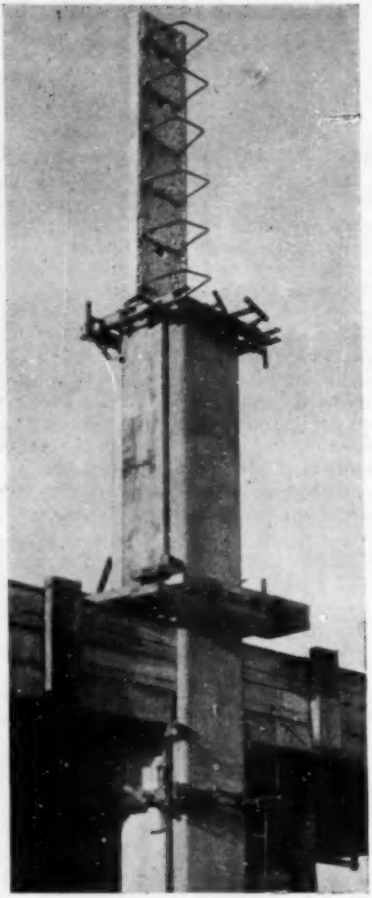


Fig. 25.—A Column partly Erected.

Fig. 24.—A Floor being constructed.

Fig. 26.—Twelve-story Flats with Precast Facing Panels and Composite Precast and In-situ Frame.

portable cranes. Clamping frames (*Figs.* 22 and 25) are used in erecting the columns. The frames are of steel or timber, with fixing screws. Standard steel or timber shutter-panels are used for the sides of the beams and columns not formed by the precast members.

A block of flats twelve stories high built on this system with precast wall panels is shown in *Fig.* 26.

PRESTRESSED PRECAST CONCRETE.

By Janik Ibsen.

The following notes and illustrations relate to prestressed precast concrete members now being made in Denmark in which the prestressing wires are pre-tensioned and bonded to the concrete.

The frame of the garage shown in

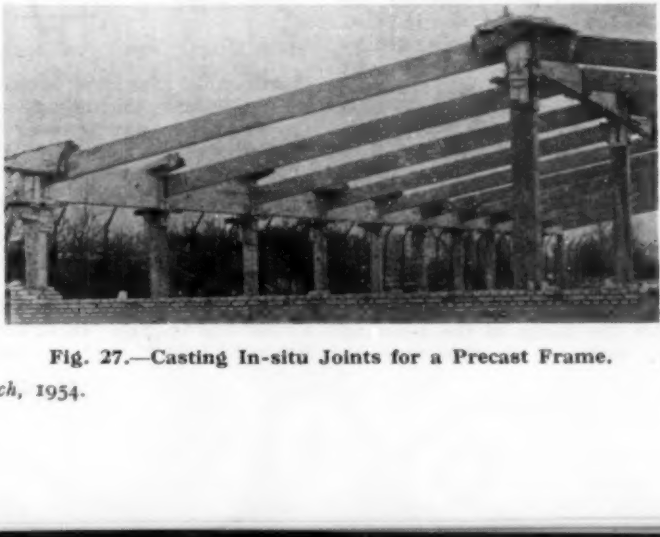


Fig. 27.—Casting In-situ Joints for a Precast Frame.

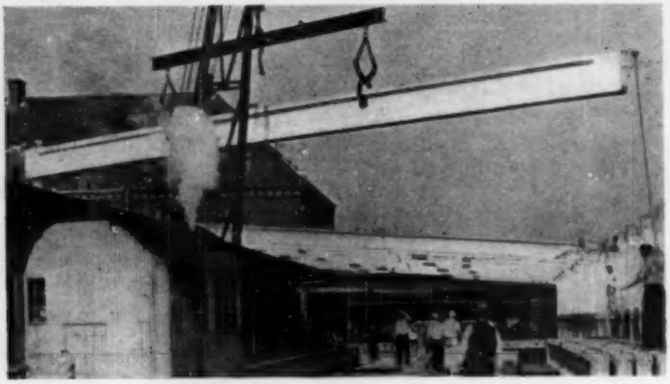


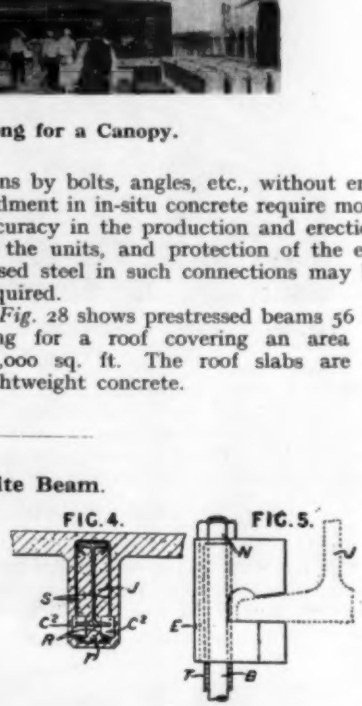
Fig. 28.—Beams 56 ft. long for a Canopy.

Fig. 27 was entirely precast. The joints were made with concrete cast in situ, and beams and columns were provided with junction bars. In such one-story buildings stability and resistance to wind pressure are most commonly obtained by fixing the columns in the footings, and the joints between the beams and the columns are designed for stresses due to changes of temperature only. Connections by bolts, angles, etc., without embedment in in-situ concrete require more accuracy in the production and erection of the units, and protection of the exposed steel in such connections may be required.

Fig. 28 shows prestressed beams 56 ft. long for a roof covering an area of 38,000 sq. ft. The roof slabs are of lightweight concrete.

Patent Composite Beam.

In a beam or rib structure of concrete cast in situ around reinforcement comprising steel bars disposed within the breadth of and below the flange of a rolled steel joist or similar steel member, drilling or slotting of the joist is avoided by positioning and interconnecting the concrete, the reinforcement, and the joist by stirrups surrounding the joist, and by clips which grip the flanges of the joist and extend laterally into the concrete. As shown in *Fig.* 4, reinforcement bars (R) are secured by stirrups (S) against the lower recessed edges of (or pass through holes in) spacing clips (C^2) engaging the lower flanges (J); the clips (C^2) may be kept in place by their own gripping action or by wire ties received in holes or in grooves (E, *Fig.* 5) therein. The spacing clips may comprise flat plates, or doubled plates having the two parts above the flange of the I-section divergent; some of the slips may be



U-shaped in plan, the arcuate section thereof housing shutter-supporting bolts (B) which are either greased before the concrete is placed or are enclosed in sleeves (T, *Fig.* 5), these bolts being secured by nuts (N) and washers. Nuts, washers, clips, and sleeves (if any) remain in the concrete and may be used for supporting fixtures, or the holes may be plugged. Additional reinforcement (*r*) may be wired to the stirrups if required.—British Patent No. 652,735. H. C. and J. O. C. Ritchie. July 1, 1947.

Prismatic Roofs with Small Angular Change.

By A. J. ASHDOWN, A.M.I.Struct.E.

(Concluded from January and February numbers.)

PRESTRESSED PRISMATIC ROOFS.

Prestressing the edge-beam in the last example would not be economical, but for a span of, say, 40 ft. it would have the advantage of reducing considerably the longitudinal stresses and the deflection of the whole structure.

As the edge-beam only can conveniently be prestressed, the effect of prestressing, due to the compressive force applied to the beam, will be to produce tensile stresses in the upper slabs; these stresses should be checked to ensure that they are not excessive.

The Influence of a Prestressing Force V.

Neglecting for the moment the continuity of the slabs and assuming that the joints are hinged, then, at the centre of the span, from (5)

$$\left.\begin{aligned} f_0\frac{a_1}{3}+\frac{f_1a_1}{6} &= V\left(\frac{1}{6}+\frac{e}{d_1}\right)\\ f_0\frac{a_1}{6}+f_1\frac{(a_1+a_2)}{3}+f_2\frac{a_2}{6} &= -V\left(\frac{e}{d_1}-\frac{1}{6}\right)\\ f_1\frac{a_2}{6}+f_2\frac{(a_2+a_3)}{3}+f_3\frac{a_3}{6} &= 0\\ \text{and, due to symmetry,}\quad f_2\frac{a_3}{3}+\frac{2f_3a_3}{3} &= 0 \end{aligned}\right\} \quad . \quad . \quad (34)$$

where V is the thrust and e is its eccentricity below the centroid of the edge beam.

Let $V = 100$, and $\frac{e}{d_1} = 0.3125 = \frac{7.5}{24}$.

Then

$$\begin{aligned} \tfrac{1}{3}f_0+\tfrac{1}{6}f_1 &= 47.92.\\ \tfrac{1}{6}f_0+\tfrac{11}{12}f_1+\tfrac{7}{24}f_2 &= -14.58.\\ \tfrac{7}{24}f_1+\tfrac{7}{6}f_2+\tfrac{7}{24}f_3 &= 0.\\ \tfrac{7}{12}f_2+\tfrac{7}{6}f_3 &= 0. \end{aligned}$$

The solution of these equations is $f_0 = 169.47$, $f_1 = -51.38$, $f_2 = 14.68$, $f_3 = -7.34$ units per square foot.

Assuming that the disposition of the cables at the ends of the edge-beam produces a resultant force through the centroid of the beam, then $\frac{e}{d_1} = 0$, and

$$\begin{aligned} \tfrac{1}{3}f_0+\tfrac{1}{6}f_1 &= 16.67.\\ \tfrac{1}{6}f_0+\tfrac{11}{12}f_1+\tfrac{7}{24}f_2 &= 16.67.\\ \tfrac{7}{24}f_1+\tfrac{7}{6}f_2+\tfrac{7}{24}f_3 &= 0.\\ \tfrac{7}{12}f_2+\tfrac{7}{6}f_3 &= 0. \end{aligned}$$

The solutions are $f_0 = 44.46$, $f_1 = 11.11$, $f_2 = -3.175$, $f_3 = 1.5875$ units per square foot.

For an example of a prestressed prismatic roof, let us use the section of the previous example, but with the span increased to 40 ft. The longitudinal stresses will be increased approximately in the ratio $\frac{40^2}{30^2} = \frac{16}{9} = 1{\cdot}\dot{7}$. Having determined the influences of a prestressing force of $V = 100$, it is possible by proportion to find the prestressing force required.

Due to the working load the stress in the bottom edge of the beam is about $-1{\cdot}\dot{7} \times 608{\cdot}1 = -1081{\cdot}1$ lb. per square inch. For the condition of zero stress at the bottom under working load, there will be at the centre of the span a stress of $169{\cdot}47 \div 144 = 1{\cdot}1769$ lb. per square inch due to a thrust from the cables of 100 lb. Therefore the thrust required is $\frac{1081{\cdot}1}{1{\cdot}1769} \times 100 = 91{,}860$ lb. If two cables each comprising twelve 0·2 in. wires are used at an allowable stress of 120,000 lb. per square inch, the total force will be

$$24 \times 0{\cdot}0314 \times 120{,}000 = 90{,}432 \text{ lb.}$$

Then, at the centre of the span, the stresses in lb. per square inch will be approximately

$$f_0 = -1081{\cdot}1 + \frac{90{,}432 \times 169{\cdot}47}{144 \times 100} = -16{\cdot}8\,;$$

$$f_1 = 172{\cdot}3 \times 1{\cdot}\dot{7} - 6{\cdot}28 \times 51{\cdot}38 = -16{\cdot}3\,;$$

$$f_2 = 33 \times 1{\cdot}\dot{7} + 6{\cdot}28 \times 14{\cdot}68 = 150{\cdot}8\,;$$

$$f_3 = -3{\cdot}48 \times 1{\cdot}\dot{7} - 6{\cdot}28 \times 7{\cdot}34 = -52{\cdot}3.$$

At the end of the span the maximum tensile stress will be

$$f_2 = -3{\cdot}175 \times 6{\cdot}28 = -19{\cdot}94 \text{ lb. per square inch.}$$

These approximations show the stresses to be quite small.

It is now necessary to find the effect of the prestressing force on the transverse bending moments B and the subsequent modification of the longitudinal stresses f. Where applicable the constants and coefficients in the previous example are multiplied by $1{\cdot}\dot{7}$, and the effect of prestressing is added. The constant in the first general equation then becomes $-25{,}650 \times 1{\cdot}\dot{7} + 904{\cdot}32 \times 47{\cdot}92 = -2265$, and in the second equation $10{,}850 \times 1{\cdot}\dot{7} - 904{\cdot}32 \times 14{\cdot}58 = 6095$. The angle of torsion in the fifth equation is also multiplied by $1{\cdot}\dot{7}$, thereby altering the coefficient of B_e and the constant. The general expressions are then as in *Table* II.

The solution of these equations gives, in lb. per square inch, f_0, 23·82 ; f_1, − 59·4 ; f_2, 212·93 ; f_3, − 106·4 ; and in ft.-lb., B_e, − 90·74 ; B_2 − 211·41 ; B_3, − 145·39. As f_3 is a fairly large tensile stress a small amount of longitudinal reinforcement should be used towards the centre of the span near the ridge.

TABLE II.

f_0	f_1	f_2	f_3	B_e	B_2	B_3	= Constant
48	24	0	0	-16·44	16·44	0	-2265
24	132	42	0	30·02	-42	11·93	6095
0	42	168	42	-25·49	71·73	-46·22	22,603
0	0	84	168	23·86	-92·45	68·59	7417
13·76	-25·06	21·28	-9·975	1232·7	1792	0	-483,293
13·76	-34·93	59·91	-38·63	896	3584	896	-950,000
0	9·99	-38·56	28·59	0	1792	3584	-912,000

The maximum torque on the edge-beam, from (24), is

$$T_0 = \tfrac{40}{8}(-2 \times 90{\cdot}74 - 169) = -2336{\cdot}5,$$

and as $T_0 = Kfbd^2$, the maximum shearing stress due to the torque is

$$f = \frac{2336{\cdot}5}{0{\cdot}281 \times 0{\cdot}5 \times 4 \times 144} = 29 \text{ lb. per square inch.}$$

The shearing force from the load on the edge beam is

$$\int_0^{\frac{L}{2}}\left(456 - \frac{B_e - B_2}{l_1}\right)dx = 20\left(456 - \frac{120{\cdot}67}{6{\cdot}06}\right) = 8722 \text{ lb.}$$

The force T_1 from (3) is $T_1 = \frac{a_1}{6}\left(\frac{M_1}{Z_1} - 2f_1 - f_0\right)$,

where $\quad \dfrac{M_1}{Z_1} = \dfrac{8722 \times 40 \times 6}{4 \times 0{\cdot}5 \times 4 \times 144} = 1817$ lb. per square inch.

Then $T_1 = \frac{144}{6}(1817 + 118{\cdot}8 - 23{\cdot}82) = 45{,}888$ lb. The shearing stress at the top of the beam is $q_1 = \dfrac{\delta T}{b\delta x} = \dfrac{45{,}888 \times 4}{40 \times 12 \times 6} = 63{\cdot}7$ lb. per square inch, and at the centroidal axis $q_m = \dfrac{3}{2} \times \dfrac{8722}{6 \times 24} - \dfrac{63{\cdot}7}{4} = 74{\cdot}9$ lb. per square inch.

The compressive stress at the centroid due to prestressing is

$$0{\cdot}5(44{\cdot}46 + 11{\cdot}11) \times 62{\cdot}8 = 174{\cdot}5 \text{ lb. per square inch,}$$

and at the top of the beam is $0{\cdot}5(11{\cdot}11) \times 62{\cdot}8 = 34{\cdot}85$ lb. per square inch.

The principal tensile stress at the centroidal axis, from $p = \dfrac{p_1}{2} - \sqrt{\dfrac{p_1^2}{4} + q^2}$, is $p = \dfrac{174{\cdot}5}{2} - \sqrt{7612 + (29 + 74{\cdot}9)^2} = -48{\cdot}4$ lb. per square inch, and at the top of the beam

$$p = \frac{34{\cdot}85}{2} - \sqrt{303{\cdot}6 + \left(\frac{29}{4} + 63{\cdot}7\right)^2} = -53{\cdot}5 \text{ lb. per square inch.}$$

Both the principal tensile stresses are small, and no special reinforcement is required. The shearing stress at the end of slab 2 where it joins edge-beam $1 = 53{\cdot}7 \times \frac{6}{3} = 127{\cdot}4$ lb. per square inch.

For slab 2,

$$P_2 = \left(320 - \frac{B_2 - B_e}{6{\cdot}06} - \frac{B_2 - B_3}{6{\cdot}89}\right)2{\cdot}88$$

$$= (320 + 19{\cdot}91 + 9{\cdot}58)2{\cdot}88 = 1006{\cdot}53 \text{ lb. per ft.}$$

Then $\quad M_2 + \Delta M_2 = 1006{\cdot}53 \times \dfrac{1600}{8} = 201{,}306$ ft.-lb.

and $\quad \dfrac{M_2 + \Delta M_2}{Z_2} = \dfrac{201{,}306 \times 6 \times 4}{49 \times 144} = 684{\cdot}7$ lb. per square inch.

$$T_2 = \frac{252}{6}(684{\cdot}7 - 425{\cdot}86 + 59{\cdot}4) = 13{,}366{\cdot}5 \text{ lb.},$$

$$q_2 = \frac{\delta T_2}{t\delta x} = \frac{13{,}366{\cdot}5 \times 4}{40 \times 3 \times 12} = 37{\cdot}13 \text{ lb. per square inch,}$$

and $$q_m = \frac{3}{2} \times \frac{100{\cdot}6{\cdot}5 \times 20}{3 \times 84} - \frac{1}{4}(37{\cdot}13 + 127{\cdot}4) = 78{\cdot}69 \text{ lb. per square inch.}$$

The compressive stress due to the prestressing force is

$$\left(\frac{11{\cdot}11 - 3{\cdot}175}{2}\right)6{\cdot}28 = 24{\cdot}9 \text{ lb. per square inch,}$$

and the principal tensile stress is

$$\frac{24{\cdot}9}{2} - \sqrt{155 + 6192} = -67{\cdot}2 \text{ lb. per square inch.}$$

This slab thus requires no special shear reinforcement.

For slab 3,

$$P_3 + \Delta P_3 = 130 \times 20 + 20\left[2{\cdot}53\left(\frac{B_2 - B_c}{l_2} + \frac{B_2 - B_3}{l_3}\right) - 5{\cdot}74\left(\frac{B_3 - B_2}{l_3}\right)\right]$$

$$= 2600 + 20[2{\cdot}53(-19{\cdot}91 - 9{\cdot}58) - 5{\cdot}74(9{\cdot}58)] = 8 \text{ lb.}$$

$$q_m = \frac{3 \times 8}{2 \times 3 \times 84} - \frac{37{\cdot}13}{4} = -9{\cdot}2 \text{ lb. per square inch.}$$

The longitudinal stresses, transverse bending moments, and co-planar shearing stresses are shown in *Fig.* 14.

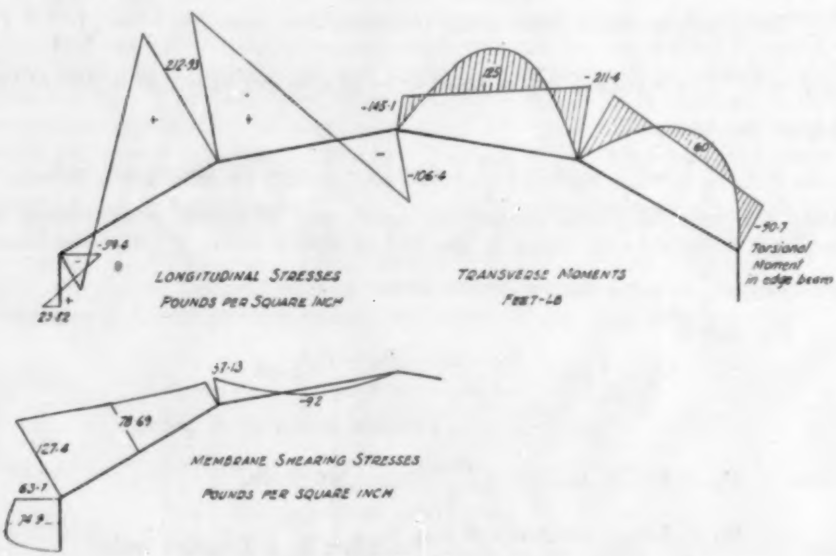


Fig. 14.

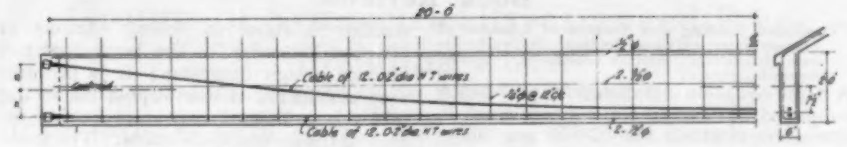


Fig. 15.

To check the final results, the vertical components of the sum of the shearing forces must be equal to the load, that is

$$a_1\Sigma q_1 + a_2\Sigma q_2 \sin\phi_2 + a_3\Sigma q_3 \sin\phi_3 = \frac{L}{2}(W_1 + W_2 + \tfrac{1}{2}W_3).$$

Then $$a_1\Sigma q_1 = \frac{(63{\cdot}7 + 4 \times 74{\cdot}9)}{6} \times \frac{6 \times 24}{20} = 435{\cdot}96 \text{ lb.},$$

$$a_2\Sigma q_2 \sin\phi_2 = \frac{(127{\cdot}4 + 37{\cdot}13 + 4 \times 78{\cdot}69)}{6} \times \frac{3 \times 84 \times 0{\cdot}5}{20} = 503{\cdot}25 \text{ lb.},$$

$$a_3\Sigma q_3 \sin\phi_3 = \frac{(37{\cdot}13 - 4 \times 9{\cdot}2)}{6} \times \frac{3 \times 84}{20} \times 0{\cdot}1736 = 0{\cdot}06 \text{ lb.},$$

and $435{\cdot}96 + 503{\cdot}25 + 0{\cdot}06 = 939{\cdot}27$ lb. and $456 + 320 + 164 = 940$ lb.

Considering the complexity of the calculations this check is excellent.

In all calculations ΔP has been assumed to be constant throughout the span. This is not strictly true as B_e increases towards the ends thereby relieving slab 2 of some of the load it is assumed to carry.

In *Fig.* 15 is shown a detail of the prestressed edge-beam with post-tensioned cables.

Resistance to Sea-water of Concrete made with Expanded-shale Aggregate.

Some information on the resistance to sea-water of concrete made with expanded-shale aggregate has been obtained by examining a concrete ship in which this aggregate was used and which, after being launched in 1919, was sunk in Galveston Bay, U.S.A., in 1922, and has remained there since. The hull was 5 in. thick.

Two 2-in. cubes cut from the hull had an average compressive strength of 10,338 lb. per square inch. Similar cubes cut from an interior rib of the ship had an average compressive strength of 8,125 lb. per square inch. At a depth of ½ in. from the surface the concrete appeared to be dry, and was not discoloured. Other cubes cut from the hull had an average compressive strength of 13,181 lb. per square inch. These strengths are considerably higher than the strengths anticipated at this age when the ship was built. Tests were made on the bond strength between the concrete and a plain round steel bar embedded for a length of 1 ft. 1 in.; failure occurred when the load was 13,175 lb., or an equivalent bond stress of 516 lb per square inch. There was no sign of corrosion of the steel.

Book Reviews.

"Combined Bending and Torsion of I-beams of Monosymmetrical Cross Section. By Ove Pettersson. (Stockholm: Swedish Cement and Concrete Research Institute.)

A BASIC equation is deduced for an I-beam subjected to bending and torsion which is symmetrical about the X—X axis only, taking to account the tendency to lateral buckling, and it is shown how this expression may be used to calculate the total resulting stress. Numerous investigators have studied aspects of the problem, usually adopting certain assumptions, but this thesis is claimed to be a new comprehensive treatment. The fundamental equation has been used to solve certain specific cases of loading; for example, when a simply-supported beam is submitted to a bending moment acting at an oblique angle to the web at one or both ends of the beam, or when a concentrated load is acting at any angle and position at the centre of a simply-supported or continuous span. The evaluation of the critical loads for lateral buckling of continuous beams symmetrical about one axis, and obtained as a deduction from the solutions of the equations for combined bending and torsion, is considered to be original. An account is given of tests to check the results of the analysis. From the complete investigation an approximate method of calculation is suggested.—J. S. T.

"The Mechanics of Engineering Soils." By P. L. Capper and W. F. Cassie. Second edition. (London: E. & F. N. Spon, Ltd. 1953. Price 25s.)

IN this edition the parts dealing with the shearing resistance and bearing capacity of soils, flow nets, and the design of roads have been rewritten. Notes on soil suction, and further examples, have been added. As was stated in our review of the first edition, this book explains the subject in a simple manner and in a way that allows the reader to judge for himself the possibility of applying the principles to the conditions with which he is concerned.

"Der Stahlbetonbau I Teil. Ausführung und Berechnung der Grundformen." By Carl Kersten, revised by Kurt Miessler. (Berlin: Wilhelm Ernst & Sohn. 1953. Price 16 D.M.)

THE nineteenth edition of this elementary textbook on the design and construction of reinforced concrete differs from previous editions in that the chapter dealing with combined bending and compression has been revised to agree with the draft recommendations of the proposed German Code of Practice. Some of the tables are similar to those in recent editions of "Beton Kalender." The book does not contain design diagrams; it is presumably considered that interpolation in the tables is sufficiently accurate having regard to the assumptions made. The great number of constructional details and examples should be of use to smaller contractors.

"Winterarbeiten im Beton- und Stahlbetonbau." By Adolf Kleinlogel. (Berlin: Wilhelm Ernst & Sohn. 1953. Price 12 D.M.)

THIS book has been revised and now records the progress made in methods of winter construction since the previous edition was published. It includes chapters on the protection of cements and aggregates, sources of heat generation and ways in which heat is lost, weather forecasting, and the cost of construction in cold weather, with many notes on practical matters. Theoretical considerations and calculations have been omitted, as it is considered that they are of little value to the man on the site.

"Building Technician's Diary, 1954." (London: Association of Building Technicians. Price 5s.)

THIS pocket-size diary comprises 120 pages of memoranda of use to those concerned with building, pages of graph paper, maps of Great Britain, as well as the diary pages.

"Verdichtungstechnik und Verdichtungsgeräte im ausländischen Erdbau." By Heinz Pösch. (Berlin: Wilhelm Ernst & Son. 1953. Price 9.80 D.M.)

DESCRIBES the methods of soil compaction developed in France, Great Britain, and the U.S.A., and the plant employed for this purpose.

Books Received.

"Studies of Slab and Beam Highway Bridges: Part III, Small-scale Tests of Shear Connectors and Composite T-beams." By C. P. Siess, I. M. Viest, and N. M. Newmark. (1952. University of Illinois Engineering Experiment Station Bulletin No. 396. Price $1.)

"Studies of Slab and Beam Highway Bridges: Part IV, Full-scale Tests of Channel Shear Connectors and Composite T-beams." By I. M. Viest, C. P. Siess, J. H. Appleton, and N. M. Newmark. (1952. University of Illinois Engineering Experiment Station Bulletin No. 405. Price $1.)

"*Alguns Problemas de Mecânica dos Solos Relativos à Parimentação de Estradas,*" by Ulpio Nascimento. "*Determinação de Tensões com Vernizes Frágeis,*" by J. L. Serafim. "*Similarity Conditions in Model Studies of Soil Mechanics Problems,*" by Manvel Rocha. "*Some Results of Settlements Observations in Actual Structures and in Models,*" by Manvel Rocha and José Folque. [Lisbon Laboratório Nacional de Engenharia Civil, Ministério das Obras Públicas. No prices stated.]

"Least-weight Proportions of Bridge Trusses." By J. L. Waling. University of Illinois Engineering Experiment Station Bulletin No. 417. (The University. 1953. Price 50 cents.)

Groups of Symmetrically-arranged Free-Standing Piles.

By C. E. REYNOLDS, B.Sc.(Eng.), A.M.I.C.E.

IN this journal for December, 1953, the writer gave the basic formulæ and some special expressions from which can be calculated the load on each pile in a group of free-standing piles containing (i) vertical piles only, (ii) inclined and vertical piles, and (iii) inclined piles only. If the piles are arranged symmetrically (*Fig.* 1) in groups of types (ii) and (iii), the summations upon which the calculations are based can be greatly simplified.

The inclined piles at a and b in *Fig.* 2 are in symmetrically arranged pairs, the two piles in each pair meeting at the same pile-cap. For this condition, if $Q_x = \dfrac{A_x}{L_x}$, the summations Σ_2 and Σ_4 referred to in the previous article are

$$\Sigma_2 = \Sigma V_x \tan\theta_x = \Sigma Q_x \cos\theta_x \sin\theta_x = 0,$$

and

$$\Sigma_4 = \Sigma V_x x_x \tan\theta_x = \Sigma x_x Q_x \cos\theta_x \sin\theta_x = 0.$$

Since the elastic modulus is likely to be the same for each pile, E is omitted from the analysis.

Consequently the distance y_0 of the centre of the system above the heads of the piles is zero, since $y_0 = \dfrac{\Sigma_2\Sigma_3 - \Sigma_1\Sigma_4}{\Sigma_1\Sigma_5 - \Sigma_2^2} = 0$. The summation Σ_3 need not be evaluated since it can readily be shown that the centre of the system is on the centre-line of the group, that is $x_0 = \dfrac{\Sigma_3\Sigma_5 - \Sigma_2\Sigma_4}{\Sigma_1\Sigma_5 - \Sigma_2^2} = 0{\cdot}5x_n$.

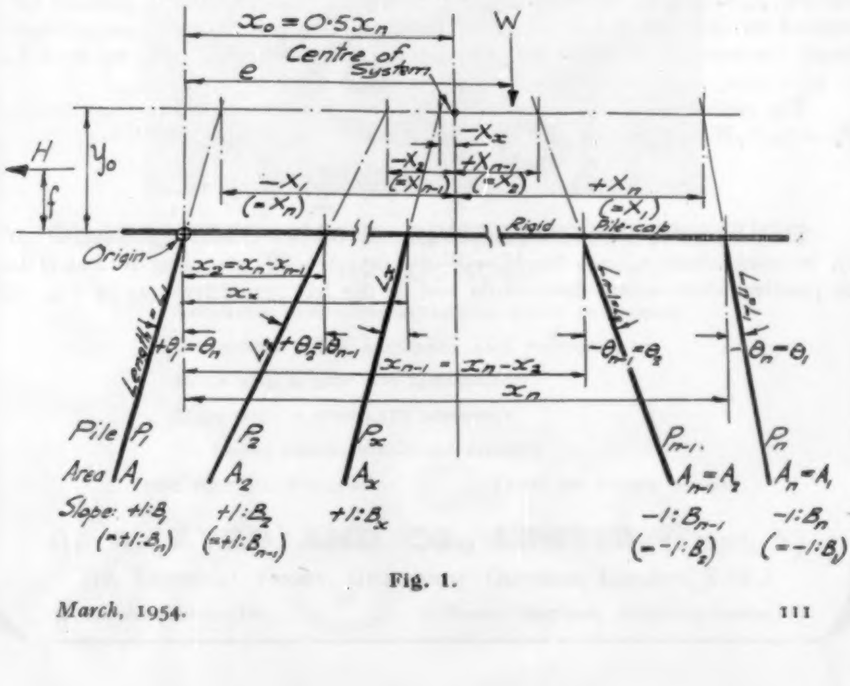


Fig. 1.

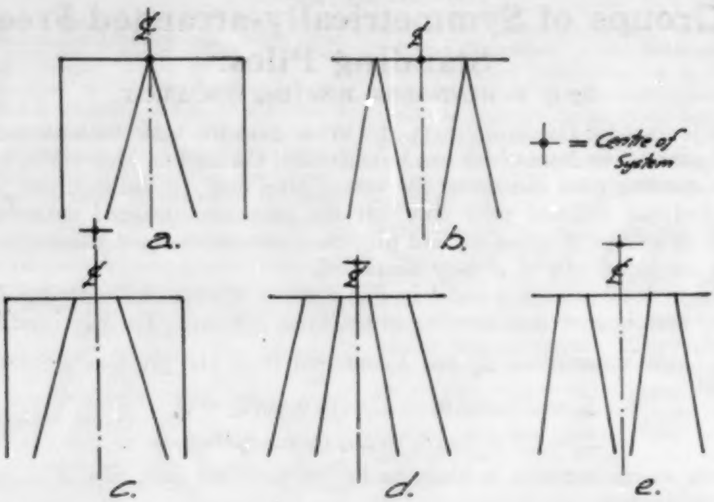


Fig. 2.

In the general case, such as *c*, *d*, and *e* in *Fig.* 2, where the piles do not meet at the pile-cap, $\Sigma_2 = 0$ but Σ_4 is not zero. Therefore $y_0 = -\frac{\Sigma_4}{\Sigma_5}$, but x_3 is still $0{\cdot}5x_n$; therefore Σ_3 need not be evaluated. The remaining summations, namely, $\Sigma_1 = \Sigma Q_x \cos^2\theta_x$, $\Sigma_5 = \Sigma Q_x \sin^2\theta_x$, and $\Sigma_6 = \Sigma X_x{}^2 Q_x \cos^2\theta_x$ need be calculated for only one-half of the group and, because of the symmetrical arrangement, the result doubled to take into account the other piles. The value of X_x is $x_x - 0{\cdot}5x_n + y_0 \tan\theta_x$, or when $y_0 = 0$, then $X_x = x_x - 0{\cdot}5x_n$.

The coefficients k in the formula for the axial load P_x on any pile, that is, $P_x = k_P(k_W W + k_H H + k_M M)$, are, for symmetrical groups, as follows:

$$k_P = Q_x \cos\theta_x\,;\quad k_W = \frac{1}{\Sigma_1}\,;\quad k_H = \frac{\tan\theta_x}{\Sigma_5}\,;\quad k_M = \frac{X_x}{\Sigma_6}.$$

The net moment M on the group is given by $M = W(e - 0{\cdot}5x_n) + (y_0 - f)H$ or, in cases where $y_0 = 0$, by $M = W(e - 0{\cdot}5x_n) - fH$, assuming W and H to be positive when acting downwards and to the left respectively as in *Fig.* 1.

A New Type of Prestressed Pile.

THE pile shown in *Fig.* 1 consists of precast hollow members, 1 ft. 4 in. square in cross section, which are assembled end to end and prestressed to form piles up to 100 ft. long. The hollow cross section is chosen because, compared with a solid pile of the same weight, it has a modulus of section about double that of a solid pile. Thus, compared with a prestressed solid pile, the prestressing force required to resist the stresses due to lifting may be reduced and the working load is therefore greater. For a given length the hollow piles can be made lighter than solid piles and are thus easier to lift and cheaper to transport. The advantages of making a pile of short members are that piles of any length can be made so that stocks of each length of pile are not required, and that the members are comparatively light and they may be made adjacent to the site.

The components of a pile are shown in *Fig.* 1. The head is solid and reinforced with ¼-in. links at 2-in. centres. The anchors for the cables are split collets bearing on a mild-steel anchor-block which in turn bears on an anchor-plate cast in the head. (The anchor-block is loose and fixed immediately before prestressing.) The anchors end 3 in. below the top of the head. It is intended to form a hole 2 ft. from the top of the head to secure the pile in the usual manner whilst driving; this is not shown in *Fig.* 1. The weight of the head is 620 lb.

The member under the head is similar to the intermediate members, but the core is bullet-shaped instead of cylindrical. The arched diaphragm transfers the forces from the solid head to the hollow members. The weight is 790 lb.

The intermediate members (*Fig.* 2) are reinforced with ¼-in. diameter helices at 9-in. pitch. The spigot-and-socket joint is to assist in fitting the members together. The diaphragms stiffen the members, and, if the head of a pile is cut off, the top member can be filled with concrete and mild steel bars cast in to tie it to a capping beam. The duct in each corner is arranged to ensure a least cover of 1½ in. to the cables; this could be increased to 2 in. if required. The weight is 720 lb.

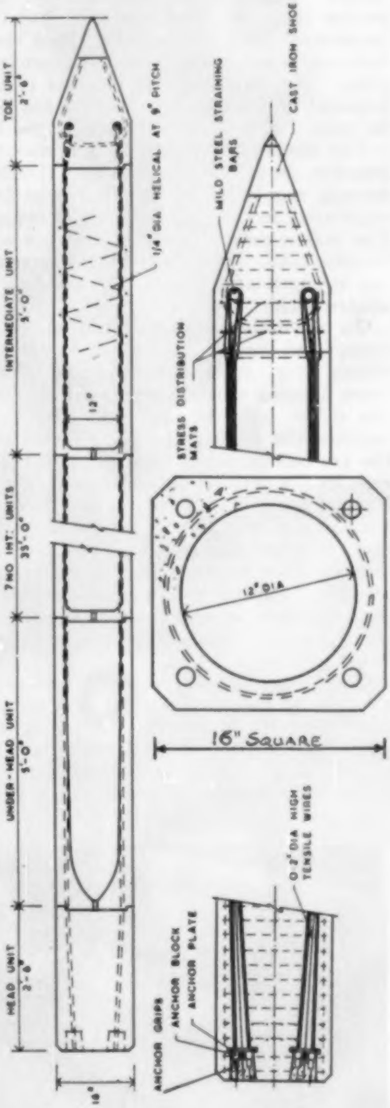


Fig. 1.—Details of Pile.

The toe serves as a fixed anchorage for the cables, which are looped around a mild steel bar. The cast-iron shoe has two loops of mild steel instead of the usual straps. These loops pass around the anchor bar, thus tying the shoe into the prestressed part of the pile. Mild steel links are used as in reinforced concrete piles. The shoes can be omitted if not required and a different type of toe can be used. The weight of a toe is 360 lb.

The members are cast in a horizontal position on a vibrating table. Timber moulds were used for the first piles but sheet-steel moulds will be used in future. The hollow members are cast with a collapsible core. The concrete has a crushing strength at 28 days of 6000 lb. per square inch.

To assemble a pile, all the members except the toe are placed end to end on rollers (*Fig.* 3), and the looped 0·2-in. wires forming a cable are passed through the ducts towards the head. In each member the ends of the ducts facing the toe are bell-mouthed to assist in threading the wires. A waterproofed cardboard ferrule (*Fig.* 4) is inserted at each joint around the cable. The ducts in the head member converge towards the top, and a spacer between the head and the member beneath it prevents the wires from bunching on the inside of the curve, which would prevent proper bonding when grouted.

The joints are successively mortared and placed together (*Fig.* 5), the ferrule preventing the mortar from entering the ducts. The toe is placed in alignment and the loop of wires passed through the square duct and secured with a mild steel bar. The joint is then mortared and the toe pushed into position.

Fig. 2.—Intermediate Members.

Fig. 3.—Assembling a Pile on Rollers.

A steel anchor-block about 3 in. diameter by 1 in. thick, and with up to eight holes arranged circumferentially to receive the wires, is threaded on to the cable, with the ends of each loop on opposite sides of the block. The block also has a central hole tapped to receive a ⅞-in. bar. The cables are in lengths varying by about 3 in. so that each loop can be identified. The anchor-block is then pushed into the socket and four anchor-grips are placed in position. By means of a screw-jack a uniform prestress of about four tons is applied to the pile, thus drawing the joints together and ensuring contact between the units. The full prestress is applied the following day. At present the jack tensions two wires only at a time, but a jack is being designed which will enable the wires to be tensioned in two operations.

The surplus length of cable is cut off and the ducts are grouted from the head, surplus grout escaping through holes. The sockets are filled with concrete and finished flush with the top of the pile. A steel bolt is screwed in the threaded central hole of the anchor-block.

Provision is made for lengthening a pile by screwing a mild steel bar into the central tapped hole in the anchor-block. The bar is thus linked to the prestressing cables and can resist its full load without splicing. Links can be provided and the pile extended as required by casting the extra length in situ.

Load-carrying Capacity and Allowable Lengths.

As the compressive stress in the concrete due to prestressing has to be

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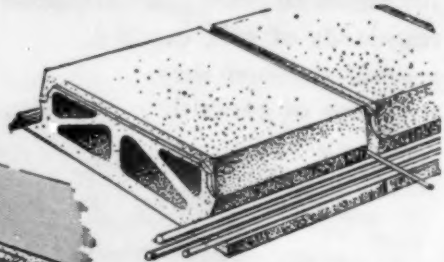
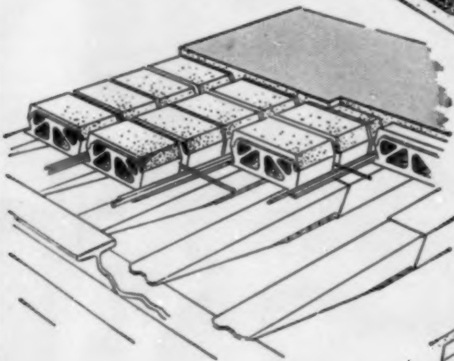


Showing Two-way Reinforcement and Hollow Concrete Blocks laid on Trianco Telescopic Centers.



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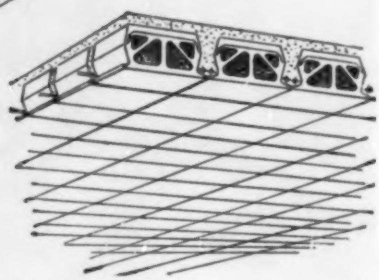

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Fig. 4.—Inserting the Cables (Waterproofed Cardboard Ferrules are used at the Joints).

Fig. 5.—Mortaring the Joints.

deducted from the allowable compressive stress under load, economies can be made by controlling the method of handling. For example, a pile 70 ft. long with a sling at one-fifth of its length from each end requires only sixteen wires and the working load is 94 tons; if the pile is slung from a single point one-third of the length from either end, then 32 wires are required and the working load is reduced to 64 tons. For piles slung at two points, each one-fifth of the length from each end, the numbers of wires required, the allowable length in feet, and the working load in tons are as follows: 16, 70, 94; 20, 80, 86; 24, 85, 79; 28, 95, 71; and 32, 100, 64.

The piles, for which a patent application has been made, are manufactured by Stent Precast Concrete, Ltd.

Two test piles 1 ft. 4 in. by 1 ft. 4 in. in cross section by 50 ft. long, with a hollow core of 12 in. diameter, have been driven. They were prestressed with twenty-four 0·2-in. diameter high-tensile wires with a breaking strength of 100–110 tons. A drop-hammer of $2\frac{1}{2}$ tons was used.

In pile No. 1 the intermediate units were reinforced with $\frac{1}{4}$-in. diameter helical binding at 9-in. pitch. The penetration was 57 ft. 6 in., the last penetration being $\frac{1}{2}$ in. for ten blows falling 4 ft.

Pile No. 2 was similar to No. 1 except that the helical binding was omitted. An accident when pitching the pile resulted in the concrete being broken from a corner at the top to such an extent that the duct was exposed for about 6 in. This pile was driven to a set of 1 in. for eleven blows of the hammer falling 4 ft. Driving stopped when the pile projected 1 ft. 9 in. from the ground and it was lengthened by 6 ft. with high-alumina cement concrete. Driving was started again twenty hours after the extension was cast, and finished when the top of the extension was 1 ft. below ground level, when the pile sunk $\frac{3}{8}$ in. for ten blows of the hammer falling 3 ft. 6 in.

New Bridge at Singapore.

THE Director of Public Works, Singapore, Mr. A. Wear, B.Sc., M.I.C.E., informs us that his department is undertaking the construction of a reinforced concrete road bridge 2,000 ft. long over the Rochore and Kallang rivers. The width of the deck is to be 65 ft., providing dual carriageways with cycle tracks and footpaths. Tenders will shortly be invited and a preliminary report describing the project and the site investigations is available from the Crown Agents for the Colonies, Millbank, London, S.W.1, or from the Director of Public Works, Singapore.

The Fire Resistance of Columns.

Tests carried out by the Building Research Station to determine the influence on the fire resistance of reinforced concrete columns of four factors, namely, variations of the load, of the strength of the concrete, and of the size of a column designed in accordance with general practice, and the effect of a very high percentage of longitudinal reinforcement, are reported in "Investigations on Building Fires. Part VI. The Fire Resistance of Reinforced Concrete Columns," by Dr. F. G. Thomas and Mr. C. T. Webster (H.M. Stationery Office. Price 3s.)

The 36 columns tested varied in size from 8 in. square to 19 in. square. The longitudinal reinforcement generally comprised four bars, but in some cases eight bars were used; the lateral reinforcement consisted of square binders. Ordinary Portland cement and gravel aggregates were used, proportioned to give cubes with crushing strengths at 28 days of 2250 lb. to 5000 lb. per square inch. The columns were stored at a temperature of 64 deg. F. and a relative humidity of 65 per cent. for periods varying from seven months to two years and nine months.

Results obtained from tests carried out in the years 1938 and 1939 are also considered. These tests were similar to those described except that some of the columns were made with gravel, limestone, and slag aggregates.

Method of Testing.

The columns were tested in accordance with the procedure specified in British Standard No. 476 except that they were loaded until failure took place and the water test and subsequent re-loading were omitted. An hydraulically-operated loading frame was used to apply an axial compression, the load being maintained constant during the test so that expansion took place freely. After the load was applied the halves of a gas-heated furnace were lighted and brought together to encircle the column over a length of 10 ft. The temperature in the furnace was measured by six thermocouples placed 3 in. from the surface of a column and the temperatures at various positions within the column were measured by thermocouples cast in the concrete. During the tests the columns could be observed through windows in the furnace.

Results and Conclusions.

The magnitude of the load had a marked effect on the endurance period of a column ("endurance period" is defined as the period between the start of the test and the collapse of the column). For example, a column designed for a safe load of 100 tons had an endurance period of about 1 hour 10 minutes when the applied load was 150 tons compared with an endurance period of nearly six hours for a similar column with an applied load of 30 tons. This period also increased with an increase in the strength of the concrete. The tests carried out to determine the influence of the size of a column did not indicate any general trends. Spalling of the arrises was marked in the larger columns and less noticeable in the smaller columns. After spalling, and when the main bars were directly exposed to the hot gases, the strength of the bars decreased rapidly. This may be considered disadvantageous when the columns have a high percentage of steel, but the results do not confirm this as in these cases eight bars were generally used of which four were at the middles of the sides where the cover remained in position for a longer period than at the corners. This effect would, however, become less important as the period of exposure to fire increased. One of the columns had a light mesh reinforcement wrapped around the main bars before the concrete was cast and the mesh, by keeping the cover of concrete in place, increased considerably the fire resistance of this column. The use of limestone or blast-furnace slag as aggregates considerably increased the fire resistance.

The investigation showed that a tentative relationship could be established between the applied load, the strength of the concrete, the size of the column, the amount of reinforcement, and the fire resistance. This may be expressed as

$$P_t = \alpha_t\,(0{\cdot}65uA) + \beta_t t_7 A_c,$$

in which P_t is the load that can be supported after a certain time in a standard fire test, u is the crushing strength of concrete cubes, A and A_c the cross-sectional areas of concrete and steel, t_7 the yield stress of the steel, and α_t and β_t are coefficients, tentative values of which are given in the report.

Some views of the open-air swimming pool at the Skegness Holiday Camp.
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The Use of Machines and the Cost of Building.

THE following is abstracted from an address given recently by Mr. Ronald W. Wates to the Institute of Quantity Surveyors.

Cranes and Hoists.

In the post-war development of plant the greatest progress has been in lifting devices, and on many contracts mobile cranes of 2 tons to 5 tons capacity are used; the design of the precast house being erected by the speaker's firm was based on the largest unit that could be handled by a crane of 10 tons capacity. In multi-story flats successful use was made of the Scotch derrick type of crane, and the French type of tower crane was a great advance upon other types. On a recent contract [see this journal for September 1953, page 307] the concreting of floors, and walls 7 in. thick, was carried out with the aid of a tower crane at a labour cost of 9*s.* per cubic yard, or about half the cost of the barrow-and-hoist method. The cost of hiring and erecting the crane, laying tracks, dismantling on completion, electricity, and driver's wages amounted to about £50 a week, but where the volume of the work warranted its use this crane was economical. These cranes also enabled large shutters, about 12 ft. by 8 ft., from floor to ceiling to be used. The walls were concreted in lifts of 8 ft. with a very great saving in labour, reduction of construction joints, and considerable speeding up of the work.

Among other types of equipment which had been developed since the war were light mobile hoists, power-driven barrows, small dumpers, and concreting-booms fitted to mixers. Many of these were labour saving and economical. For example, using a concreting boom, the concrete in house foundations and rafts cost as little as 7*s.* per cubic yard for mixing and placing in position. Other developments were the use of site-generated electricity for small power-driven tools, vibrators, and drills. A recent development was the bolt-gun which, by means of an 0·22-in. cartridge, fired a bolt through steel and into concrete; on one contract about 35,000 bolts were used to fix steel windows to concrete frames and the saving of labour was considerable.

Loose Cement.

The more general use of loose cement was an important development and less expensive than cement delivered in bags. His firm used a storage silo erected on the site; the cement was brought in a tank vehicle fitted with compressed air plant which blew the cement through a flexible hose from the lorry to the silo.

Auger Piles.

Power-driven short auger piles seemed the best and cheapest method of providing deep foundations to houses where these were necessary on clay sites. On a housing contract in the Midlands the foundations were to have been supported on piers at an extra cost of about £30 per house, but by using power-driven auger piles at 7 ft. centres the cost was reduced to about £10.

The Cost of Building.

It is almost axiomatic to say that the greatest productivity arises where the maximum possible power per man is employed, and there is a close relationship between speed of production and economy. At their present level, building costs were about the most that people would pay for commercial or industrial buildings or houses. Based on a 1939 basis of 100, the current cost of living was said to be 164. According to the Girdwood report, the average cost per square foot of a house built by a local authority in 1939 was 9*s.*, and to-day the average cost was not less than 33*s.* While the cost of living had risen about 1·64 times, the cost of building had risen about 3·6 times. Costs had been fairly stabilised during the last year despite the increase in wages. World commodity prices showed a tendency to fall and this was bound to affect prices. The productivity of building labour was slowly rising. Another factor which would help to cheapen building would be the elimination of rise-and-fall clauses in contracts. He was in favour of increasing the responsibility of the builder in this matter, as he believed that the builder was better able than anyone else to get fixed prices from his merchants and thereby set in train a greater sense of responsibility throughout the work.

Partnership.

Mr. R. W. Hawkey, sole partner of the firm of Sir Cyril Kirkpatrick & Partners, has joined Mr. E. O. Measor and Mr. H. Grace, partners in the firm of Scott and Wilson, and they are now practising under the title of Scott and Wilson, Kirkpatrick & Partners. Mr. F. M. Bowen has also been taken into partnership. From February 15 the address of Messrs. Scott and Wilson, Kirkpatrick & Partners will be 47 Victoria Street, London, S.W.1 (telephone Abbey 6255; telegraphic address Pontifact, Sowest, London).

IMPERIAL COLLEGE OF SCIENCE AND TECHNOLOGY—DEPARTMENT OF CIVIL ENGINEERING

Bursaries in Concrete Technology

NOTICE IS HEREBY GIVEN that the election to Bursaries in Concrete Technology tenable as from October, 1954, will take place in June, 1954.

Candidates must hold a degree in Engineering at the time of taking up the award, and must also have a good knowledge of the theory of structures.

Bursaries are of the value of £350 per annum, out of which the College Tuition Fee has to be paid; the amount may be increased to £450 for those with industrial experience. In addition, one or two Senior Bursaries of £600 per annum may be awarded to outstanding men with a minimum of three years' experience in industry.

The course will be postgraduate and Bursars who successfully complete the course will be eligible for the award of the Diploma of the Imperial College (D.I.C.).

Applications must be received on or before June 1st, 1954, by the Deputy Registrar, City and Guilds College, Exhibition Road, London, S.W.7, who will, on written request, send full information and application forms.

DATA FOR PRICING REINFORCED CONCRETE.

Materials.

(Delivered in London area.)

AGGREGATES (per cu. yd.). — Washed sand, 22s. 3d. Clean shingle: ¾ in., 18s. 11d.; ½ in., 21s. 6d. Pit ballast, 21s. 5d.

CEMENT (per ton, delivered at Charing Cross).—
- Portland cement, 6 tons and upwards, 95s. 1 ton to 6 tons, 104s. Paper bags and non-returnable jute sacks included.
- Rapid-hardening Portland, 8s. above ordinary Portland.
- Aquacrete and 417, 32s. 6d. above ordinary Portland; paper bags included.
- Colorcrete (buff, red, and khaki), in 6-ton loads, 136s. 6d.; paper bags included.
- Snowcrete, £12 15s. 6d., inc. paper bags.
- "Super-Cement," 32s. 6d. per ton above ordinary Portland cement; paper bags included.
- High-alumina cement, 1 ton and upwards, 280s. 9d. per ton.
- Snowcem paint, 71s. per cwt. inc. containers.

SHUTTERING.—For prices of timber, refer to S.R. & O., 1949, No. 1079 (price 1s. 1d.) and No. 94 (price 5d.) issued by H.M. Stationery Office.

REINFORCEMENT.—Mild steel bars, B.S. 785 (per cwt.); 1 in. to 2½ in., 42s. 3d.; ⅝ in. to ⅞ in., 43s. 9d.; ½ in., 44s. 3d.; ⅜ in., 45s. 9d.

Materials and Labour.

(Contracts up to £5000. Inc. 10 per cent. profit.)

PORTLAND CEMENT CONCRETE, 1 : 2 : 4.—
- Foundations, 2s. 4½d. per cu. ft. Columns, 3s. 3d. per cu. ft. Beams, 3s. per cu. ft. Floor slabs 4 in. thick, 8s. 7d. per sq. yd.; Do., 5 in., 10s. 9½d.; Do., 6 in., 12s. 4d.; Do., 7 in., 14s. 5½d. Walls 6 in. thick, 13s. 9½d. per sq. yd. Add for hoisting above ground level 3s. 7½d. per cu. yd. Add for rapid-hardening Portland cement 2s. 6d. per cu. yd.

REINFORCEMENT.—Mild steel bars (B.S. 785), including cutting, bending, fixing, and wire (per cwt.)—¼ in. to ⅜ in., 72s. 9d. 7/16 in. to 1 in., 67s. 1⅛ in. to 2½ in., 61s. 6d.

SHUTTERING AND SUPPORTS.—
- Walls, 203s. 6d. per square.
- Floors (average 10 ft. high), 188s. 9d. per square. In small quantities, 2s. 6d. per sq. ft.
- Columns, average 18 in. by 18 in. (per sq. ft.), 2s. 10½d.; in narrow widths, 3s. 7½d.
- Beam sides and soffits, average 9 in. by 12 in. (per sq. ft.), 2s. 9½d.; in narrow widths, 3s. 3½d.
- Raking, cutting, and waste, 5¾d. per lin. ft.
- Labour on splays, 2¾d. per lin. ft.
- Small fillets to form chamfers, 5¾d. per lin. ft.

Wages.

The rates of wages on which the above prices are based are: Carpenters and joiners, 3s. 9d. per hour (carpenters 2d. a day tool money); Labourers, 3s. 3½d.; Men on mixers and hoists, 3s. 5½d.; Bar-benders, 3s. 6d.

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MISCELLANEOUS ADVERTISEMENTS.

Situations Wanted, 3d. a word : minimum, 7s. 6d. Situations Vacant, 4d. a word : minimum, 10s. Other miscellaneous advertisements, 4d. a word : 10s. minimum. Box number 1s. extra. The engagement of persons answering these advertisements is subject to the Notification of Vacancies Order, 1952.

Advertisements must reach this office by the 23rd of the month preceding publication.

SITUATIONS VACANT

SITUATION VACANT. Consulting structural engineer, Westminster, requires senior designer-draughtsman with first-class experience in structural steelwork and reinforced concrete, for responsible position. High salary and good prospects for suitable applicant. Write in confidence, stating age, qualifications, and full details of experience. Box 4007, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Consulting structural engineer, Westminster, requires experienced reinforced concrete draughtsman-detailer. High salary and good prospects for suitable applicant. Write, stating age, qualifications, and full details of experience. Box 4006, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Assistant civil design engineer required by Dorman, Long & Co., Ltd., for design work in connection with extensive development schemes in progress and planned. Reinforced concrete experience essential. Applicants must be capable of developing schemes and supervising their subsequent detailing and progress. Apply, stating age, experience, salary required, and when at liberty, to CHIEF TECHNICAL ENGINEER, CENTRAL ENGINEERING DEPARTMENT, DORMAN, LONG & CO., LTD., G.P.O. Box 11, Royal Exchange, Middlesbrough, Yorks.

SITUATIONS VACANT. Reinforced concrete designers required for Midlands office of specialist firm. Men with at least five years' experience required. Apply stating age, experience, and salary required. Five-days' week. Staff canteen. Box 462, 19/21 Corporation Street, Birmingham, 2.

SITUATION VACANT. Experienced reinforced concrete designer-detailer for small firm of consulting engineers in London. Experience of structural steelwork an advantage. State age, education, experience, and salary required. Box 4010, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Assistant engineer, £750 per annum basic salary, required by small London firm of consulting engineers. Experience in reinforced concrete structural design with basic civil engineering training and site experience. Able to take responsibility. Opportunity for tours abroad. Box 4011, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Designer-draughtsman required for London office of well-known reinforced concrete engineering contractors. Experience in reinforced concrete frames, floors, roof, and staircase construction essential. Progressive post. Pension scheme. Alternate Saturdays. Write fully Box 295, ALLARDYCE PALMER, LTD., 109 Kingsway, London, W.C.2.

SITUATIONS VACANT. JOHN LIVERSEDGE & ASSOCIATES, civil and structural engineers and consultants, require immediately the following additional engineering staff. (a) Civil-engineer-designers, fully qualified and experienced in all types of reinforced concrete work, including design of industrial and shell concrete structures, and with some structural steelwork experience. Salary range up to £1500 per annum. (b) Reinforced concrete designer-detailers and detailer-draughtsmen. Salaries according to capabilities. Modern office conditions. Normally five-days' week. Apply to 42 Portland Place, London, W.1, for form of application.

SITUATION VACANT. Designer-draughtsman for reinforced concrete, Westminster office, capable of preparing complete designs for reinforced concrete structures, including blocks of flats, offices, shops, etc. Able to work with minimum supervision. Pension scheme. Alternate Saturdays free. Box 4014, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

DRAUGHTSMEN

SIMON - CARVES LTD.

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at their Cheadle Heath and Manchester offices. Applications are invited from draughtsmen with sound experience of heavy mechanical and structural work, particularly of mechanical handling and steel-framed buildings. Working conditions and scope are excellent, and a D.O. bonus scheme, pension fund, and non-contributory sickness benefit scheme are in operation. Apply in writing, quoting ref. (QS 44), giving age and experience to :

**STAFF & TRAINING DIVISION,
SIMON-CARVES LTD.,
CHEADLE HEATH, STOCKPORT.**

SITUATIONS VACANT. Several experienced reinforced concrete detailers required. Salary according to qualifications and experience. Five-days' week. Staff canteen. Apply "TWISTEEL" REINFORCEMENT, LTD., Alma Street, Smethwick, Staffs.

SITUATION VACANT. Assistant civil engineer required in London office of a firm of architects and consulting engineers. Applicants must be qualified, capable of directing a team, and have had several years' experience in the design and detailing of both steel and reinforced concrete structures. Knowledge of other branches of civil engineering will be an asset. Applications are to give full particulars of training and experience, and are to state the salary required. Box 4015, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Qualified structural and/or Civil engineer for technical sales with a London firm of precast concrete specialists in piling and structural units. Wide connections desirable and permanency guaranteed to a suitable engineer. Pension scheme in operation subject to certain conditions. Initiative and drive required, capable of producing superior results. A four-figure salary will be payable plus additional payment by results, together with reasonable expenses. Replies, which will be treated in the strictest confidence, must state in chronological order general experience, together with age, salary required, if in possession of a car (which is desirable), and when available to commence duties. Present employers need not be mentioned until interview granted, but present position must be stated, together with names of two referees. Box 4016, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. AIR MINISTRY WORKS DEPARTMENT requires in London structural engineer designer/draughtsmen experienced in reinforced concrete or structural steelwork. Applicants should have sound technical training and several years of varied experience in design and detailing of (a) reinforced concrete construction for all types of buildings, or (b) steel-framed sheds, warehouses, and kindred types of buildings. Salaries up to £733 per annum, starting pay dependent upon age, qualifications and experience. Extra duty allowance or overtime payable. Reasonable prospects of promotion. Posts temporary and non-pensionable, but have long-term possibilities. Competitions held periodically to fill established vacancies. Applications from natural born British subjects only, quoting EE and stating age, qualifications, and previous appointments, giving dates and stating type of work done, to AIR MINISTRY, S.2(h)/NA, Cornwall House, Stamford Street, London, S.E.1.

(Continued on page 120.)

Building Research.

THE following are abstracts from the Report of the Building Research Board for the year 1952 (recently issued by H.M. Stationery Office. Price 3*s.* 6*d.*).

Bridge Decks.

Strain measurements are being made on the girders of a new filler-joist bridge of 28 ft. span carrying the Great North Road across the railway at Biggleswade and designed to satisfy the new heavy-load requirements of the Ministry of Transport. The measurements have been proceeding since April, 1952. It was found that the stresses due to dead load corresponding to the measured strains were much smaller than the stresses obtained by the usual methods of calculation. This is partly because the deck was cast in two operations. The first layer was supported by the joists alone, and after the first layer had hardened the second layer was supported by the joists and the first layer acting together. It is proposed to carry out loading tests on this bridge with heavy vehicles.

A test to destruction on a slab-and-girder model bridge deck with encased girders has been completed. Its ultimate load-bearing capacity with a concentrated load directly over a girder was about half-way between those of models with and without shear-connectors but with no concrete encasement of the girders. The mode of failure was similar to that of previous slabs, that is a truncated cone was pushed out of the slab under the load. In the bending tests there was a tendency for the loaded girder, with a portion of the slab reaching in width to the two adjacent girders, to separate from the rest of the structure. Failure occurred as a result of yielding of the bottom flange of the loaded girder, followed by failure in shear at the supports.

Prestressed Concrete.

Experimental work has been done on the fire resistance of composite beams consisting of a prestressed concrete beam of rectangular or I section and a reinforced concrete slab cast on later to form a tee section. Three sizes of each type of beam were made to determine the influence of size on the fire resistance, and some were protected with vermiculite concrete. With gravel aggregate and Portland cement, and with hard-drawn wire, failure of a loaded beam became imminent when the temperature of the steel exceeded 400 deg. C. Failure due to a reduction in strength of the steel appears to occur more frequently than failure due to a reduction in strength of the concrete. Spalling of the concrete leading to a premature collapse, which may result when small prestressed beams of slender cross section are exposed to fire, did not occur. It is estimated that a fire resistance of two hours could be obtained with 2½ in. cover of concrete to the steel. Greater resistance could probably be developed with a thicker cover. For the highest grades of fire resistance, exceeding four hours, the tests have shown that additional protection is effective in delaying the rise of temperature in the cable.

Investigation of the effects of steam curing at atmospheric pressure on production on "long-line" prestressing beds has shown that the loss of prestress in this method of curing due to changes in temperature during manufacture is unlikely to exceed 12 per cent. for concrete made with gravel aggregate.

Strain gauges were installed in a prestressed concrete office building at Kilburn to obtain information on its behaviour during construction and subsequently over a period of years. The strains recorded in the prestressing operations were in reasonable agreement with those calculated. The strains recorded during loading tests on completion of the framework showed, however, that the stiffness of the main beams was appreciably greater than had been assumed in design, owing to composite action with the floor slab. [See page 91 of this number.]

Frames.

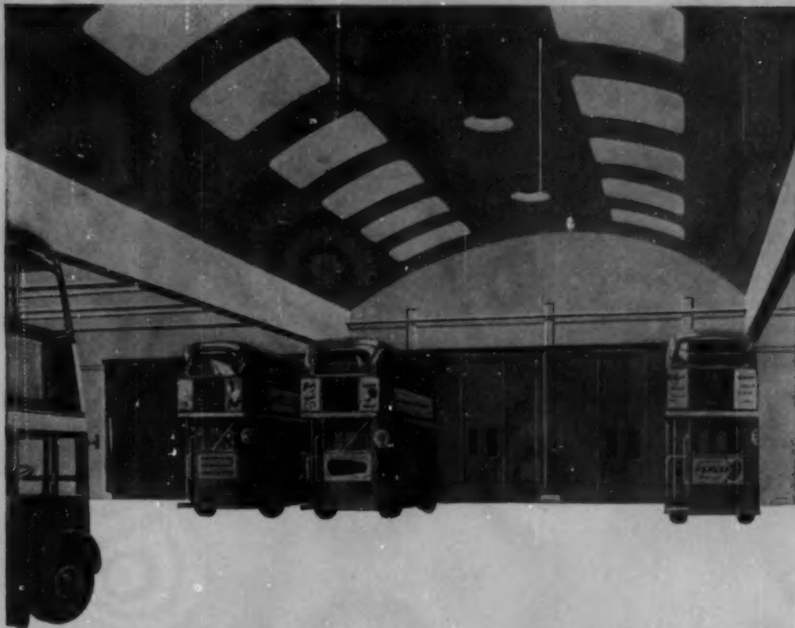
Tests on framed structures have been started to ascertain how the filling between multiple-story frames helps to stiffen and strengthen the frame and to resist racking action resulting from lateral forces such as wind pressure.

In one test the stiffness of an encased-steel frame was increased considerably even with a relatively weak filling such

as might be used for a partition. The maximum racking load sustained with a $4\frac{1}{2}$-in. brick filling was over twice that which could be supported by the frame alone, and much of the help afforded by the brickwork was still retained when a door opening was left in the panel. It was evident from the tests that allowance for the racking strength of walls and partitions might often avoid the provision of special connections or bracing for resisting lateral forces in structural frameworks.

Economy of Steel.

A study has been made of the quantities of steel actually used in comparable portions of eleven recently constructed schools representing several types of structural frames. The object was to ascertain the weights of steel per square foot of comparable floor area. It appeared that the design most economical in steel would comprise load-bearing walls, prestressed concrete main beams for floors and roof, prestressed concrete or reinforced concrete floors (if timber is not available), and a non-ferrous roof. In practice, however, the maximum economy of steel may not always be possible, because of factors such as speed and ease of erection, labour problems, and supply difficulties.



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